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JANUARY
1950

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VOLUME 76
No. 1

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TWO PARTS
PART 1

PROCEEDINGS OF THE SOCIETY

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 76 JANUARY, 1950 No. 1, PART 1

TECHNICAL PAPERS

AND

DISCUSSIONS

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GUIDEPOST FOR TECHNICAL READERS

JANUARY, 1950, *PROCEEDINGS*

Items of value or significance to readers in various fields are here listed. For convenience the distribution is given in terms of the Society's Technical Divisions

(A constant effort is made to supply technical material to Society members, over the entire range of possible interest. In so far as your specialty may be covered inadequately in the foregoing list, this fact is a gage of the need for your help toward improvement.—Ed.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

DESIGN CHARACTERISTICS OF LOCK SYSTEMS IN THE UNITED STATES

A SYMPOSIUM

Note.—Written comments are invited for publication; the last discussion should be submitted by June 1, 1950.

FOREWORD

The lifting or lowering of vessels vertically from one water level to another by locks is not new. The earliest boatmen resorted to unloading, carrying, and reloading to overcome distance and differences in elevation between bodies of water. Necessity brought about the development of inclined planes and the use of waves resulting from controlled releases of stored water. The forerunners of the modern lock, possibly in the year 1000, consisted of single gates installed in dikes along the North Sea, where the fluctuations of tide were employed to lift and lower shipping. Records indicate that actual "chamber locks" were invented about the same time in Germany, Holland, and Italy, perhaps near the end of the fifteenth century. The great Italian engineer Leonardo da Vinci, about 1497, invented the insertion of filling valves and emptying valves in the lock gates, which system has been used for centuries and has been found adequate for the passage of small boats.

Today, there are in the United States locks in active operation which are not greatly different from those designed by da Vinci and also there are many which have been designed on the basis of exhaustive laboratory testing and years of experience in operating locks which pass many millions of tons of traffic each year.

It is the purpose of the Committee on Design, Construction, and Operation of Navigation Structures of the ASCE Waterways Division, which prepared the Symposium, to present papers setting forth the latest thought regarding the design of locks and other navigation structures. The present coverage, admittedly, is incomplete, but it is hoped that it will be a start toward publication of design bases which have not recently been presented to the engineering profession, and which are well known to a relatively few individuals.

It is to be hoped that discussion of these papers will bring to bear on the problems the thoughts of a larger group of engineers, producing new ideas on all specific subjects presented herein, and perhaps suggesting to the committee the most needed direction for future papers and discussions.

INFLUENCE OF MODEL TESTING ON LOCK DESIGN

By A. Frederick Griffin,[1] M. ASCE

Synopsis

Hydraulic systems of various types in use in locks on many American rivers and canals are described briefly, together with observations as to the manner in which they operate. The objectives of engineers engaged in lock design are stated and the important part played by the hydraulic laboratory as an aid in determining ways and means of improving the efficiency of lock-filling and lock-emptying systems is depicted. Some general criteria for guidance in design, based on results of comparatively recent prototype and model testing, together with examples of designs produced in conjunction with laboratory testing, are presented. The writer finds that the locks reported are used effectively in the transportation of traffic, yet practically none is operated as designed. Through experience, a valve-operation procedure has been developed for each lock which provides satisfactory lockage conditions. However, operating time has been increased in varying degrees over that planned in the design, and the over-all efficiency of the lock has been correspondingly decreased. Lock hydraulic systems should be designed so they can be operated as planned. The writer suggests that it is both possible and practicable to design systems that will provide fast and safe operation without creating intolerable conditions within the lock chamber or in the lock approaches. Lock models have been found efficient and practical in resolving many pertinent difficult hydraulic problems. Much in the field remains to be accomplished and the continued use of models is strongly recommended.

Introduction

The development of lock hydraulic systems has been largely predicated on economy of time and of materials—that is, on the value of time lost during lock operations balanced against the additional cost of expenditures required to minimize surges, turbulence, and adverse currents. As long as barges and towboats were small and slow moving, the value of the time expended in lifting or lowering vessels while passing from one pool to another was not consequential. Small craft and small tows could be accommodated in lock chambers so small that surges and turbulence were not too troublesome. Filling and emptying systems consisting of valves in the lock gates are both inexpensive and, for low-lift locks of small size, satisfactory. However, progress in the development of shipping and barge transportation, the size and draft of vessels and of barge tows, the increase in the speed and power of vessels, the volume of water transportation, and the size and lift of locks—have all con-

[1] Chf. Engr., Office of Div. Engr., Upper Mississippi Valley Div., Corps of Engrs., U. S. Army, St. Louis, Mo.

tributed to the need for further development, and perhaps refinements, in the operating features of navigation locks.

It is the purpose of this paper, through references to various locks of which the writer has direct or indirect knowledge, to call attention to: (*a*) The features of a variety of hydraulic systems used or proposed; (*b*) observed performance at representative locks; (*c*) some prototype and model tests employed to discover methods of improving the operating characteristics of lock hydraulic systems built or contemplated; and (*d*) the practical use of lock models as a preliminary to lock design. Much has been written by numerous observers and investigators on various phases of the subject of lock hydraulic systems. In general, such papers have dealt with specific problems of the subject. In this paper an effort is made to summarize and to generalize, with a view toward affording the profession a compendium and discussion of salient features of modern lock design and performance. Obviously, such a treatment results in the inclusion of much information which is neither new nor novel, but which is considered essential for a balanced treatment of such a broad subject.

Types of Lock-Filling and Lock-Emptying Systems

Lock systems currently in use may logically be classified into the following basic types:

A. Those filling and emptying over, between, or around the lock gates;

B. Those filling and emptying by valves in the gates, or through short culverts around the gate bays, through the lock walls, or in the gate sills; or

C. Those filling and emptying through longitudinal culverts in the walls or floor, connected to the lock chamber through either wall ports or floor laterals.

Type. A. Filling and Emptying Over, Between, or Around the Lock Gates.— With lock gates of the sector or Tainter type it is possible to eliminate the need for special filling and emptying valves and conduits and to use the lock gates for this purpose. Sector gates (see Fig. 1) are in general use as lock gates where

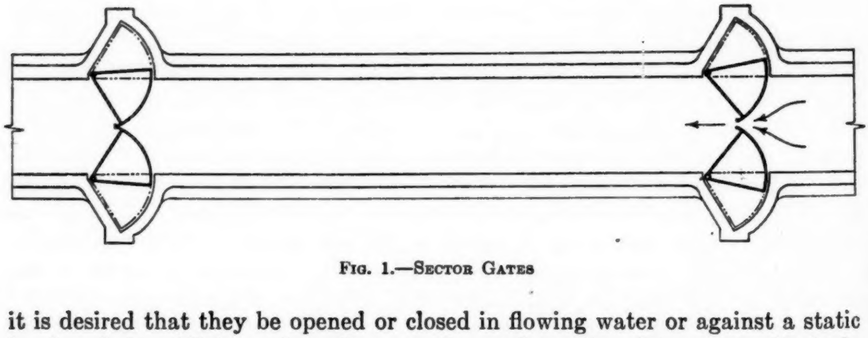


Fig. 1.—Sector Gates

it is desired that they be opened or closed in flowing water or against a static head and, in addition, where they can be used to fill and to empty the lock. This type of gate consists of two leaves, each composed of a sector of a vertical cylinder which rotates about a vertical axis at or behind the face of the lock

wall. The two leaves abut against one another at the center line of the lock when closed, and are withdrawn into recesses in the lock walls when opened. They were developed for use on the Södertalje Canal in Sweden, where it was desired to pass storm water through the lock. In the United States, sector gates have been used in the lock at the mouth of the Chicago River in Illinois, where differentials in water levels cause reversals of flow; in Bayou Sorrel Lock at the junction of the Intracoastal Waterway and the East Atchafalaya Basin Protection Levee, in Louisiana; in the St. Lucie Locks in Florida; and on several large drainage canals. To fill a lock equipped with sector gates, water is admitted between the edges of the gate leaves and between the leaves and the upstream corners of the wall recesses. It has been found by experiment or testing that, if the major part of the flow is through the recesses and if the curve of the recess walls directs the stream from each side into the chamber in a direction perpendicular to the axis of the lock, turbulence in the lock chamber is not excessive for relatively small heads.

The application of Tainter gates for use as lock gates has not been extensive. They are receiving consideration as upper lock gates, however, in connection with the design of several proposed locks. The Tainter gate would be opened for lockages by lowering below the lock sill and would provide for filling by flow over the top of the gate. Model tests indicate that turbulence and surge in the lock chamber can be controlled to a reasonable extent by using baffles between the gate trunnions and by employing the upstream end of the lock floor as a stilling basin. Provision of satisfactory lockage conditions for medium lifts appears feasible.

Type B. Filling and Emptying by Valves in the Gates, or Through Short Culverts Around the Gate Bays, Through the Lock Walls, or in the Gate Sills.—Most of the locks constructed in the late nineteenth century and early twentieth century on the Ohio River in Pennsylvania and bordering Ohio, West Virginia, Indiana, Kentucky, and Illinois; the Monongahela River in Pennsylvania; the Cumberland River in Tennessee and Kentucky; and other rivers were of this type. The system consists of either valves integral with the lock gates or a number of short culverts passing directly through the river wall, or through or around the gate sill, each controlled by a separate valve. Filling and emptying by valves in the miter gates may be exemplified by the Deep Creek and South Mills Locks in the Dismal Swamp Canal near the Virginia–North Carolina state line. These locks are small—52 ft by 300 ft. They have a lift of about 12 ft and were designed to accommodate movement of small craft. Turbulence in the upper end of the lock chamber and both turbulence and high velocities experienced below the lower gates have not been found detrimental to the type of traffic using the waterway.

Many of the locks on the Ohio River are of low lift and are filled or emptied through a series of ports in the river lock wall controlled by butterfly valves (see Fig. 2). The filling valves, in the upstream section of the wall, discharge directly from the upper pool into the lock chamber. The emptying valves, in the downstream section of the same wall, discharge directly into the lower pool. Filling time averages from about 5 min to 8 min, and emptying requires from 4 min to 6 min, depending on the head. During the filling operation, flow

through the filling ports creates a turbulent eddy in the lock chamber with a downstream flow along the lock land wall. Somewhat similar currents are experienced during the emptying operations. These conditions are not too noticeable for the lower lifts but are troublesome when the lift approaches from 8 ft to 10 ft. The action of the currents described is ordinarily controlled by judicious opening of the filling and emptying valves, the times for filling and emptying for lockages being lengthened ordinarily by as much as 3 min.

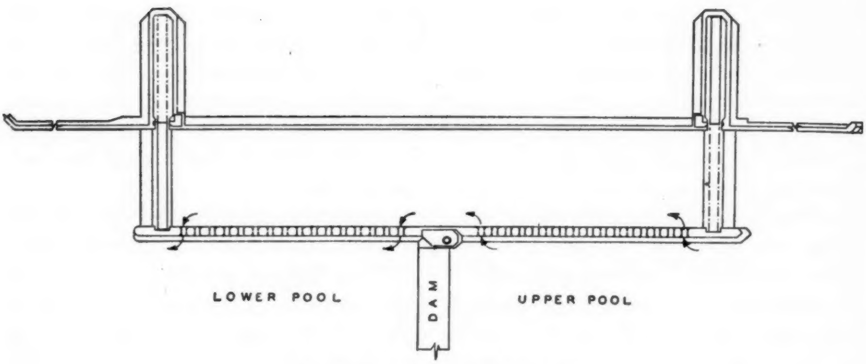


FIG. 2.—VALVES IN RIVER WALL

Locks on the Monongahela River (where the hydraulic systems consist of short culverts either around or in the upper sill and valves in the lower gates) are of relatively low lift and it is reported that hydraulic operations have not resulted in undue turbulence or in other conditions adverse to traffic.

On the Cumberland River, filling valves are located in culverts under the upper miter sill. Lifts for the 52-ft by 280-ft locks vary from 5.5 ft to 19.5 ft. Filling time is normally from 5 min to 8 min. Turbulence in the locks is not severe but it is general practice in filling to use a one-third valve opening until the lock chamber is one third full, a two-thirds opening until the lock chamber is two thirds full, and a complete opening for the remainder. This practice results in slightly longer filling periods.

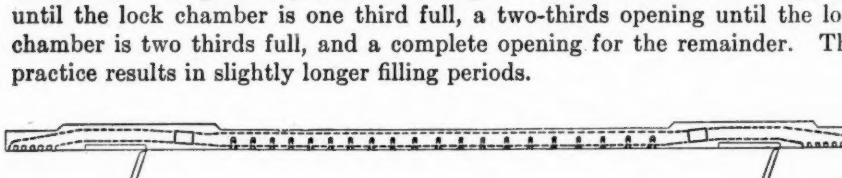


FIG. 3.—"CONVENTIONAL" TYPE

Type C. Filling and Emptying Through Longitudinal Culverts in the Walls or Floor, Connected to the Lock Chamber Through Either Wall Ports or Floor Laterals.—What may be termed the "conventional American type" of hydraulic system (see Fig. 3) consists of wall intakes in the upper approach walls, longi-

tudinal culverts in the lock walls, wall-chamber filling and emptying ports distributed more or less evenly throughout the length of the lock, and wall-discharge ports below the lower lock gates. This type is found in the more recently constructed locks on the Allegheny and the Monongahela rivers in Pennsylvania, the Ohio River, the Kanawha River in West Virginia, the Tennessee River in Tennessee and Kentucky, and the Upper Mississippi River bordering Minnesota, Wisconsin, Iowa, Illinois, and Missouri; the Illinois Waterway; the New York State Barge Canal; the Intracoastal Waterway along the Atlantic and Gulf coasts; and also in the locks in the Welland Canal in Canada. In general, the conventional system has performed satisfactorily for relatively low lifts.

Experience in the Ohio Valley

At the Gallipolis Lock on the Ohio River in Ohio and the three locks on the Kanawha River, which have "conventional" systems, the lifts vary from 24 ft to 28 ft. When filling these lock chambers in the minimum possible time, noticeable turbulence and surge action obtain. In actual practice this action is substantially reduced by incremental valve opening, the filling time being increased possibly 2 min. While emptying these locks in minimum time, pronounced turbulence and surge action are present in the lower approach, particularly just below the lower gates. It is necessary to slow down the discharge by proper manipulation of the valves to prevent the strong currents from breaking mooring lines. Minimum emptying periods for these locks are increased as much as 4 min.

At Lock No. 41 on the Ohio River, where the maximum lift is 37 ft, adverse conditions obtain in both the upper and lower approaches and in the lock chamber. (The dams on the Ohio River are numbered westward[2] beginning at a point west of Pittsburgh, Pa.; dam No. 41 is west of Louisville, Ky.). The upper approach is the Portland Canal, in which, during filling operations, current velocities of approximately 2 ft per sec and troublesome surges exist. These disturbances are sufficiently severe to necessitate special care in mooring the first sections of upbound double-lockage tows securely to avoid the possibility of barges breaking loose and damaging the upper gates. The hydraulic system for the main lock, conventional as to side culverts and lock-wall ports, deviates from the "conventional" in that the filling and emptying systems are entirely separate, the filling being restricted to the upper half of the lock and the emptying to the lower half (see Fig. 4). Excessive turbulence exists during filling operations, transverse currents are strong enough to move tows from side to side of the chamber, and longitudinal surge continues until the lock is about one third full. During emptying operations turbulent conditions of less consequence exist in the lock chamber but current velocities of from 6 ft per sec to 7 ft per sec may obtain in the lower approach. These currents and the accompanying turbulence require tows to be securely moored at a considerable distance below the lock to avoid parting of lines. Minimum

[2] "Some Notes on the Location and Construction of Locks and Movable Dams on the Ohio River, with Particular Reference to Ohio River Dam No. 18," by William M. Hall, *Transactions*, ASCE, Vol. LXXXVI, 1923, p. 94, Fig. 1.

filling and emptying times for "main lock" No. 41 are 20 min and 18 min, respectively. To overcome adverse operating conditions, at least partly, the filling and emptying times are ordinarily 27 min and 21 min, respectively, a lengthening of 7 min and 3 min in filling and emptying periods, respectively.

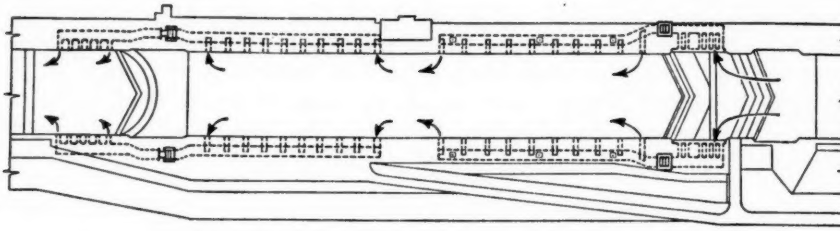


FIG. 4.—LOCK NO. 41, OHIO RIVER (LOUISVILLE, KY.)

Experience at auxiliary lock No. 41, an example of the conventional side-culvert, wall-port system, indicates that it is possible to effect substantial reductions in the time required for filling and emptying lock chambers by better designs of the filling and emptying systems.

EXPERIENCE IN THE TENNESSEE VALLEY

There are 11 locks in use on the Tennessee River,[3] ten of which operate under high lifts—that is, from 39 ft to 80 ft. All these locks have conventional type filling and emptying systems, the more modern locks offering systems modified as to spacing, size, and shape of ports in accordance with the results of model testing. Model studies were used extensively in determining the type and action of the lock hydraulic systems, some being performed at the Hydraulic Laboratory of the University of Iowa, Iowa City, and others at the Hydraulic Laboratory of the Tennessee Valley Authority (TVA), Knoxville, Tenn. Velocity tests were made to determine the size and shape of intake and discharge ports, the size of culverts, and the spacing of ports, and to obtain uniform flow into the culverts and through ports. Surge in the lock chamber, filling and emptying times, and open and closed systems—that is, vented or unvented—were investigated.

Turbulence in these locks during filling operations, although quite noticeable, is reported to have little effect on tows, but for small boats must be reduced by slowdown of the valve-opening period. Turbulence is held to a minimum by controlling the inflow until there is sufficient water over the floor of the chamber to prevent excessive disturbance. Valve-opening time for Fort Loudoun and Watts Bar locks in Tennessee is from 3 min to 4 min and results in relatively satisfactory conditions. At other locks the operator usually delays the valve-opening period by holding the valve in a partly open position for varying periods, depending on the size of the tow. Filling times are lengthened slightly but are considered satisfactory. There is a slight tendency for tows to move upstream at all locks. Transverse currents are minor. Tur-

3 "Allocation of the Tennessee Valley Authority Projects," by Theodore B. Parker, *Transactions*, ASCE, Vol. 108, 1943, p. 175, Fig. 1.

bulence below the lower gates is considerable at locks where discharge is into the lower approach, but has little apparent effect on upbound tows, which normally stay well below the gates until the chamber is ready for lockage. It would be troublesome in the event of double lockages, in all probability necessitating a considerable lengthening in emptying time.

Experience on the Mississippi River

Locks on the Upper Mississippi River, except No. 1 (in Minnesota) and No. 19 (in Iowa),[4] are 110 ft by 600 ft, with lifts varying from 5.5 ft to 25 ft. The hydraulic systems consist of the conventional longitudinal side culverts and lock-wall ports as shown in Fig. 3. Wall ports are spaced evenly throughout the length of the lock, those in one wall being directly opposite the corresponding ports in the other wall. Depth over the lock floor is only 13 ft at low-water stage. Filling and emptying times vary from 5 min to 8 min, depending on the head. Variable degrees of turbulence exist in all locks during filling and below the lower gates during emptying operations, depending on the operating head. Both transverse and longitudinal currents exist in the locks, particularly during the first stage of filling, and are more noticeable at the lower third of the chamber. Turbulence and adverse currents are reduced to suit the type of traffic by regulating and extending the time of opening valves. The increase in filling time varies from 2 min to 4 min. During double lockages, the emptying process must be lengthened by slower valve operation to diminish turbulence in the lower approach when unattended tows are moored to the lower guide wall. The increase in emptying time varies from 2 min to 6 min, depending on existing conditions. As modified, hydraulic operations are quite satisfactory.

Experience on the Illinois Waterway

The 7 locks on the Illinois Waterway are each 110 ft by 600 ft. Lifts at low water vary from 10 ft to 40 ft. The conventional hydraulic system is used. Wall ports are directly opposite each other. The average time of filling varies from 12 min to 20 min; and the time of emptying, from 9 min to 16 min. These periods, in general, result in satisfactory operations, with no noticeable turbulence in lock chambers. Transverse and longitudinal currents are negligible. However, turbulence below the lower gates at the Lockport Lock[5] (lift, 40 ft) is such that small boats and light tows are kept at least 300 ft downstream during emptying operations.

Experience on the Intracoastal Waterway

The Harvey Lock connects the Mississippi River with the section of the Intracoastal Waterway that lies west of New Orleans, La. It has a usable lock chamber, 75 ft by 425 ft. The lift varies from 0 ft to 20 ft, depending on the Mississippi River stage. The filling and emptying systems are the con-

[4] "Canalization of the Upper Mississippi," by E. L. Daley, *Civil Engineering*, February, 1936, p. 105, Fig. 1.

[5] "Engineering Features of the Illinois Waterway," by Walter M. Smith, *Transactions*, ASCE, Vol. 98, 1933, p. 310, Fig. 1.

ventional types. Time of filling or emptying at maximum stage is 8 min. Model tests were not made in connection with the design. As originally constructed, considerable turbulence resulted in the lock chamber, together with longitudinal surge. These conditions were substantially corrected by reducing the size of the filling ports in the lower end of the chamber. Transverse currents are such that caution must be exercised in filling the lock chamber to prevent the breaking of mooring lines, particularly with smaller tows. The inflow must be regulated to have the stronger flow enter from the side to which the tow is moored. Turbulence in the lower approach is such that boats are not allowed to approach closer than 150 ft until outflow has ceased.

Experience at the Welland Canal Locks

The 7 locks in the Welland Canal overcome a lift of 325.5 ft, an average of 46.5 ft. Usable dimensions of locks are 80 ft by 820 ft. Filling and emptying systems consist of longitudinal culverts in lock walls and side ports spaced on 30-ft centers near the bottom of the walls. The depth over the sills is 30 ft. The hydraulic system as designed permits 8-min filling and emptying. The actual operation period is 15 min, during which valves are opened one fourth, one half, three fourths, and fully, during a total period of about 3 min. This slower rate of filling results in negligible surges and turbulence. Considerable turbulence obtains below the lower gates during normal discharge.

Experience at the Locks at Sault Ste. Marie, Mich.

Among the first locks in the United States was one built by the North West Fur Company, in the St. Marys River at Sault Ste. Marie. This lock was only 8.75 ft wide and 38 ft long, with a lift of 9 ft. The importance of traffic between Lake Superior and Lake Huron required enlarged facilities at this location from time to time—first by the State of Michigan in about 1853, and since 1873 by the federal government. The latter has constructed 5 locks at this location: The Weitzel Lock in 1873 to 1881 followed by the Poe Lock in 1887–1896, the Davis Lock in 1908–1914, and the Fourth Lock, 1913–1918.

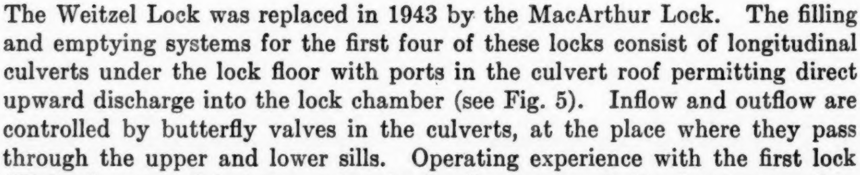


Fig. 5.—Fourth Lock, Sault Ste. Marie, Mich.

The Weitzel Lock was replaced in 1943 by the MacArthur Lock. The filling and emptying systems for the first four of these locks consist of longitudinal culverts under the lock floor with ports in the culvert roof permitting direct upward discharge into the lock chamber (see Fig. 5). Inflow and outflow are controlled by butterfly valves in the culverts, at the place where they pass through the upper and lower sills. Operating experience with the first lock (Weitzel) was satisfactory, and so hydraulic systems similar but increased in

size were developed and used in the Poe, Davis, and Fourth locks. Depths on the floor at lower pool are 16 ft, 23 ft, and 23 ft, respectively. These systems have proved to be quite satisfactory in so far as disturbance within the lock chamber is concerned. Considerable disturbance is present in the lower lock entrances as a result of direct discharge from floor ports, but this has little detrimental effect on navigation, as approaching upbound ships stay well below the gates until locks are ready for entrance and erosion is not a problem. However, a serious problem did develop in the upper approach to the locks from the surge, back and forth, in the upper canal. Waves of considerable magnitude were experienced and the channel depth was affected as much as 1.5 ft. Ships were grounded and lines of ships moored to the upper approach walls, awaiting lockage, were snapped because of the longitudinal forces during lock filling. The original times for filling and emptying the large locks were 10 min and 8 min, respectively. Valves opened at a uniform rate resulted in a rate of inflow too high for existing approach conditions—6,500 cu ft per sec. The situation and the prototype tests made to ease it were described in some detail in 1937 by Horace M. Edmands.[6] The tests resulted in operational changes so that valve openings are governed by a filling curve having equal and gradual rates of acceleration and retardation of flow, so that there is a maximum rate of inflow of about 5,600 cu ft per sec, and so that time of filling is lengthened 3 min—that is, to 13 min.

Experience on the Columbia River

The Bonneville Ship Lock in Oregon, opened to navigation in 1938, was designed with the aid of hydraulic models, which were used where practicable to test the basic design assumptions which, because of lack of precedent for high-lift locks, were necessarily based on analytical and theoretical reasoning.

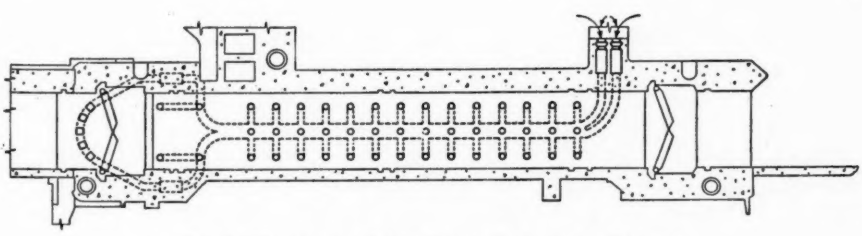


Fig. 6.—Bonneville Lock, Columbia River, in Oregon

The lock is 76 ft wide and 500 ft long. Its maximum lift is 66 ft. Its hydraulic system, shown in Fig. 6, consists of a double intake through one lock wall, a single longitudinal culvert beneath the lock floor along the center of the lock, fourteen branches thereto, and forty-two evenly distributed floor ports permitting vertical flow into the lock chamber. The minimum depth on the lock floor is about 26 ft. The discharge passes through the lock-floor system and around the lower gates via loop culverts with a valve chamber in each lock wall,

[6] "Lock Filling at St. Marys Falls Canal," by Horace M. Edmands, *The Military Engineer*, January–February, 1937, pp. 62–64.

to five vertical discharge ports in the floor of the lower approach. Normal filling and emptying take about 12 min.

Model tests indicated that the lock could be filled and emptied best with a slow valve opening at the beginning and with an accelerated movement near completion. Although the Bonneville model did not indicate excessive turbulence, subsequent model studies on a somewhat similar filling system for another lock showed individual boils of strong intensity over each floor port. This "boiling" is duplicated in the prototype to such an extent that the operating force throttles the flow, particularly when small craft are being locked. Inflow is not evenly distributed, being intensified in the lower end of the lock chamber, which causes a considerable longitudinal slope in the water surface. Concentration of discharge just below the lower gates results in a "fountain effect" that raises the water surface as much as 6 ft. A comprehensive prototype test determined the maximum line stress, when mooring the floating plant of 383 tons displacement, to be 10 tons, the maximum occurring at time of greatest rate of inflow.

Experience at the Keokuk (Iowa) Lock, Mississippi River

The 110-ft by 358-ft lock at dam No. 19 utilizes a filling system (see Fig. 7) somewhat similar to that at Bonneville. The maximum lift at this lock is 38.2 ft. The water flow enters through intakes in the upper end of the river wall and discharges vertically into the chamber through a longitudinal culvert in that wall, eight laterals beneath the floor, with seven ports in the roof of each lateral. The depth of water cushion at the beginning of the filling operation is normally about 10 ft. The minimum filling time is 15 min. With such operation, excessive turbulence is experienced, individual boils over each floor

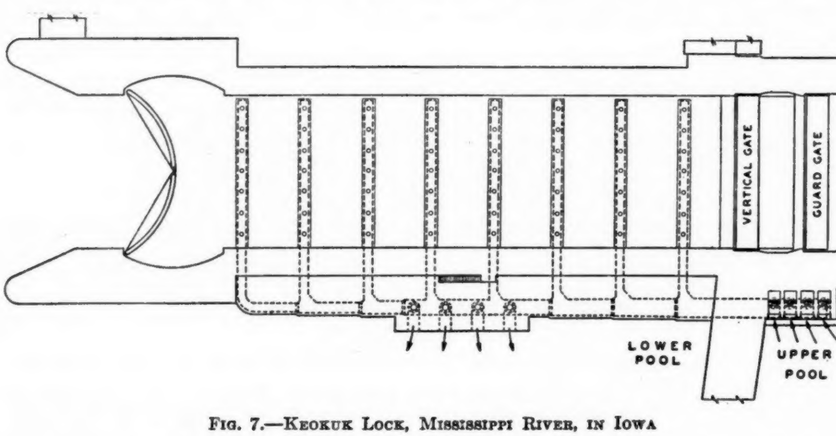


Fig. 7.—Keokuk Lock, Mississippi River, in Iowa

port mounting as much as 2 ft or 3 ft above the normal water surface. Lockage of small craft and even small tows is dangerous without judicious filling operations. The ordinary filling period is lengthened to 18 min. This lock discharges through the river wall directly to the river, thus eliminating adverse conditions in the lower approach chargeable to lock discharge.

Summary of Experience

The locks reported in this paper are used effectively in the transportation of traffic. However, practically none is operated as designed, particularly in the case of upbound lockages or when sections of tows are moored to the lower guide walls. Through experience, a valve-operation procedure has been developed for each lock which provides satisfactory lockage conditions. However, in doing so, the operation time has been increased from 50% to 100% more than that planned in the design.

Design Objectives

The objectives of those engaged in lock design have been, and are, to design the lock hydraulic system so as to:

(1) Permit filling or emptying the chamber in the briefest period compatible with unobjectionable turbulence, line stress, crosscurrents, or surges within the lock chambers;

(2) Minimize disturbances in lock approaches, thus permitting more efficient use of guide walls for mooring vessels: and

(3) Obtain an even distribution of inflow and discharge across the lock entrances to reduce the concentration of velocities, surges, eddies, and erosion, giving due consideration to the economics of the solution.

The Hydraulic Laboratory "Enters the Picture"

The complexity of a lock hydraulic system is such that it does not lend itself well to rational analyses; and, in consequence, the hydraulic laboratory has played an important part in determining how to improve the efficiency of lock-filling and lock-emptying systems. In recent years models of lock hydraulic systems have been used extensively to predict operational characteristics of the prototype, to check the design before construction, and to study operating conditions of existing structures. It has been found practicable to reproduce, in the model, effects and conditions comparable to those that will be experienced in the prototype. The practicability of models was demonstrated in 1923 and 1924, in the River Hydraulics Laboratory at Karlsruhe Germany, by identical tests on the model with a scale ratio of 1:50, of a group of locks at Steenenhoek, Netherlands, on the Merwede Canal, and on the prototype. Comparison of observations of water level, discharge, and slopes determined the mean deviation for sixteen sets of observations to be but 3.5%.

In the United States similar tests were run on Wheeler Lock in Alabama, the first project completed by the TVA. The model was constructed in 1936 to a scale ratio of 1:20 with exact geometric similarity to the prototype that was then in operation. Observations were made on both the prototype and model of the (*a*) water level in the lock chamber, (*b*) velocities and discharges from the lock-chamber ports, (*c*) flow conditions in the intake ports, and (*d*) pressures in one of the culverts. In addition, photographic records were obtained of the turbulence in the lock chamber and in the discharge section. Comparison of results shows a reliable agreement between the model and prototype in filling time, slope of the water surface in the lock chamber, and

maximum values of rise and fall of the water surface. The emptying time for the prototype was 8% less than that for the model, which reflects a higher hydraulic efficiency in the prototype system. The model did show that the behavior of the prototype could be predicted with reliability.

Additional tests were obtained by the Division Engineer of the Ohio River Division on the Guntersville (in Alabama) and Pickwick (in Tennessee) locks of the TVA for verification of model tests to determine the reliability of the model as a unit or of its individual parts. The results of such tests, together with observations and the limited prototype tests made on the MacArthur Lock at the "Soo," have indicated such excellent agreement between model and prototype that is is felt the reliability of the hydraulic laboratory work in the study of the more complex phases of lock hydraulic systems has been established.

Although hydraulic tests have been made on lock-emptying and lock-filling systems in many hydraulic laboratories in the United States, such testing (pertaining largely to the side-culvert or floor-culvert systems and their modifications) has been more or less concentrated with the laboratories of the federal government at Knoxville, Bonneville, Iowa City, Vicksburg, Miss., and Balboa Heights, Panama Canal Zone. Some of the projects tested at these laboratories, since 1938, are as follows:

	Name of structure tested	Location
Knoxville (TVA)	Guntersville Lock	Tennessee River in Alabama
	Gilbertsville Lock	Tennessee River in Kentucky
	Watts Bar Lock	Tennessee River in Tennessee
Bonneville (Corps of Engineers)	Willamette Falls Lock	Willamette River in Oregon
	Dam 1, Ice Harbor Site	Lower Snake River in Washington
	McNary Lock	Columbia River in Oregon
Vicksburg	Algiers Lock	Intracoastal Waterway west of New Orleans
	New Jersey Ship Canal-Locks	Intracoastal Waterway
Iowa City	Lock No. 2	Mississippi River in Minnesota
	Lock No. 19	Mississippi River in Iowa
	Lock No. 27	Mississippi River in Illinois
	New Cumberland Locks	Ohio River in Ohio
	MacArthur Locks	St. Marys Falls Canal in Michigan
Balboa Heights	Third Locks	Panama Canal

General Results of Prototype and Laboratory Tests

Experience from prototype and model studies has developed three general criteria as a guide for design, based on currents, turbulence, and conditions in approach channels:

Currents.—Longitudinal currents in lock chambers, created by rapidly accelerated inflow, may be minimized if the first chamber port or bottom lateral is located a distance below the upper miter gate equal to from 25% to 30% of the lock-chamber length. Operation of the filling valves to eliminate a sudden inrush of water at the beginning of the filling period is important. Gradually accelerating and retarding the inflow will reduce ship movement and line stress, but at the expense of filling time. In practice lock operators use this method to reduce line stresses.

Uniform distribution of inflow along the lock walls will do much to eliminate the formation of currents in the lock chamber. Prototype experience and tests in the hydraulic laboratory support theory in indicating that, in the interest of uniform distribution of flow, port or lateral spacing should be varied by decreasing the spacing distances toward the upstream end of the chamber.

Bellmouth entrances and diverging side walls of lock-chamber ports result in decreased chamber inflow velocity and improved entrance flow conditions without decreasing the port-discharge capacity. Transverse currents can be minimized in a lock chamber fed through a single side culvert by use of properly designed floor laterals.

Turbulence.—Turbulence in the lock chamber is largely the result of improper conditions for the most efficient dissipation of the energy contained in the inflow. Actual operation of prototypes shows that an important feature affecting turbulence and oscillating surges is the time required for valve opening. If the valves are opened slowly enough, no turbulence will develop and surging will be reduced. Of course, a longer filling time results. A bottom lateral system will reduce turbulence under a quick valve opening and thus permit a shorter filling time.

Prototype tests indicate that the safe rate of inflow for side-filling systems is a function of the water-cushion depths in the lock chamber. The greater the depth the larger will be the rate of inflow that may be accommodated without causing undue turbulence and surges.

A decided improvement in side-port-filling operations can be realized by staggering the location of ports in one wall with respect to those in the opposite wall. This action permits adjacent jets to pass each other and to travel the entire width of the lock before being deflected, rather than to impinge on each other in the center of the lock, thus effecting greater diffusion and dissipation than are possible when ports are located on coincident center lines. Transverse flows are less pronounced and the residual surface flow is toward the center of the lock.

The entrainment of air in the inflow causes severe turbulence in the lock chamber as the air is expelled from the system in large bubbles under pressure. The culverts should preferably be placed well below the elevation of the lower pool, particularly for high-lift locks, so that the hydraulic gradient is above the roof of the culvert, to reduce the amount of air entrained during the valve-opening period and to minimize the possibility of valve chatter. In addition, the tendency for air to be entrained in a system equipped with Tainter valves can be minimized by facing the skin plate downstream from the trunnions and sealing the downstream stop-log recess.

A larger culvert area (about 20% larger) is required in a bottom-filling system to obtain the same operation schedule as from an efficient side-culvert system.

Turbulence in a lock chamber, particularly where the depth of water over the floor of the lock is small or where locks are subject to relatively high lifts, can be reduced by the use of floor laterals discharging horizontally through side ports. The floor laterals should be placed with the upstream lateral about 25% or 30% of the lock-chamber length downstream from the upper lock gate. The laterals should be spaced to provide a distance between them approximately equal to the width of a lateral. Side ports in any lateral should be so spaced that they alternate with those in adjacent laterals to permit discharges to impinge on opposite walls rather than on each other.

In the more effective dispersion of the jets, the bottom-filling system has an advantage over the side-port system for locks having shallow depth over the floor. However, turbulence caused by vertical jets in a bottom-filling system augments the disturbance caused by any uneven distribution of discharge. To minimize this adverse condition, the jets should be horizontal.

Approach Channels.—In addition to turbulence and surges within the lock chamber, the characteristics of the filling and emptying systems will have a predominant influence on conditions in the approach channels. The effects are particularly aggravated where the size of the approach channels is limited, as is the case where the lock is located in a canal. Pondage in the vicinity of the upper lock entrance has been found to be a practical method of reducing the surges within the canal.

Below the lock, the discharge from the emptying system during lockage may necessitate an increase in the length of the lower guide wall because of the increase in hazards to traffic, particularly during double lockages. A diffuser in the lower entrance of a lock, immediately below the downstream miter gates, provides uniform distribution of the discharge from the lock across the lower approach, localizes the turbulence in a small area near the gates, and consequently permits more efficient use of the lower guide wall than if the water is discharged in the conventional manner through horizontal ports in the lower approach walls.

The MacArthur Lock, St. Marys River, Sault Ste. Marie

General.—To present an example of recent lock design in the United States, the MacArthur Lock will be discussed in some detail. This design was based on a general mathematical approach, available information, and experimental model data on similar systems; and refinements were made in accordance with the results of model tests in the lock-testing laboratory of the Corps of Engineers at Iowa City. The model simulated, to a scale of 1:25, the complete lock and parts of the upper and lower approach channels. In prototype dimensions the lock is 80 ft wide by 800 ft long between service gates; it has a lift of 22 ft and a depth of water cushion over the lock floor of 31 ft. The construction of the prototype was begun in April, 1942, and was completed in July, 1943. Since changes in design were anticipated, the model was constructed so that the parts would be accessible and as easily removable as practicable.

The hydraulic system adopted (see Fig. 8) consists essentially of a longitudinal culvert in each side wall of the lock, terminating with intake and discharge manifolds in the upper and lower pools, respectively; transverse lateral culverts beneath the floor of the lock alternately connected to the wall culverts; and vertical ports through the cover slab of each lateral. Flow of water through the system from the upper pool into the lock chamber and from the lock chamber into the lower pool in the filling operation and the emptying operation, respectively, is controlled by interposed valves of the reversed Tainter-gate type.

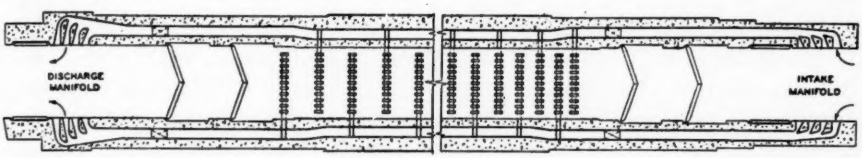


FIG. 8.—MACARTHUR LOCK, SAULT STE. MARIE, MICH.

Design Criteria.—The basic criteria that governed the design were

(*a*) The filling and emptying system was to be of sufficient capacity to fill the lock in 8 min and to empty it in 7 min at a lift of 22 ft. In actual operation the maximum rate of inflow was not to exceed about 5,400 cu ft per sec to prevent adverse surging in the upper approach.

(*b*) Disturbances in the lock chamber which would tend to increase line stresses were to be kept to a minimum.

(*c*) Any transverse slopes existing in the lock chamber should tend to hold vessels against the wall of the lock rather than to move them to the center of the lock.

These criteria were established after an investigation of the lock-approach conditions and the characteristics of the waterway traffic. The difficulties experienced in the past in the approach channel (see previous observations on the "Soo" locks) dictated that every precaution should be taken to insure that conditions would not be aggravated.

Conduits.—The approximate size of the lock culverts required to fill or to empty the lock was determined from the relation between the volume of the lock chamber, the lift, and the contemplated efficiency of the lock-filling system, with cognizance taken of the maximum permissible filling rate. This relation has been expressed as

$$A_C = \frac{2\,A_L\,\sqrt{h}}{C\,t\,\sqrt{2\,g}}$$

in which A_C is the total culvert area, in square feet; A_L is the area of lock chamber, in square feet; h is the lift, in feet; C is a coefficient—a measure of the efficiency of the filling and emptying systems; t is the time required to fill or empty the lock, in seconds; and g is the acceleration of gravity, in feet per second per second. The foregoing equation does not take into consideration either the reduced effective culvert area available during the valve-operating

period or the dynamic head operating on the system during a large part of each filling and emptying operation. Allowances were made for these factors. It was determined that, to satisfy the emptying-time and the filling-time criteria, a total culvert area of approximately 300 sq ft would be required. Although friction losses in circular culverts are somewhat less than those in rectangular culverts of equal area and roughness, in the case of lock hydraulic systems rectangular culverts permit the construction of more efficient entrance and exit sections at intake, lock chamber, and discharge ports; and, since the energy lost by turbulence at inefficient entrances and exits would greatly exceed that lost by friction, culverts, rectangular in shape and 11 ft by 14 ft in size, providing 154 sq ft of area in each lock wall, were adopted.

To minimize the possibilities of cavitation and of entrainment of air in the culverts, the tops of the culverts were placed about 14 ft below the minimum recorded tailwater, so that a positive pressure head would always exist in them. Subsequent model tests indicated that, although the pressure head in the culvert near the Tainter valve would drop somewhat below the tailwater elevation for a short period immediately after the start of the filling and emptying operations, the selected elevation would assure that positive pressure would always exist in the culverts.

Intake Ports.—Intake ports were patterned after a streamlined intake-port manifold developed as a result of previous model testing (see Fig. 8). This type of intake-port manifold had been incorporated in 2 prototype locks constructed on the Tennessee River and had performed satisfactorily. To obtain an equal distribution of inflow through each of the ports, it is necessary that the port throats be progressively decreased in width, proceeding downstream from the first port. The ratio of the total throat area of the intake ports to the total culvert area for the MacArthur Lock is 1.39.

Laterals and Lock-Chamber Ports.—Because a greater degree of turbulence is likely to exist in the laterals than in the culverts and because, as a result, a smaller proportion of the lateral area is effective than in the case of culverts, it has been found desirable to provide a total lateral area somewhat greater then the total culvert area. Consequently, twenty-four laterals, rectangular in shape and 4 ft by 4 ft in size, alternately connected to opposite culverts by an elbow-shaped transition section, were adopted, providing a total lateral area of 384 sq ft. Tests later conducted on the model, during which the amount of total lateral area was varied, indicated that the basic design in this respect was satisfactory. The ratio of lateral area to culvert area is 1.25.

There were few design data on the spacing of the laterals along the lock chamber. However, experiments conducted at the Hydraulic Laboratory of the Corps of Engineers (United States Department of the Army), at Iowa City, on a model of a side-port filling and emptying system indicated (and checked the theory) that during the filling operation the pressure gradient in the culvert would have an upward slope in the direction of flow. Consequently, a greater pressure head would exist at ports or laterals near the downstream end of the culvert and they would carry an appreciably larger volume of flow per unit of area than would ports or laterals near the upstream end of the culvert. To

compensate for this effect and to attain more even longitudinal distribution of flow within the lock chamber, the distances between the individual laterals were varied, laterals near the upstream end of the lock being more closely spaced than those near the downstream end.

The actual spacing of the laterals in the MacArthur Lock model was based on the data obtained from the aforementioned tests on a side-port system. Although the desirability of this type of spacing was confirmed by tests later conducted on the model, the unbalance of flow distribution proved to be somewhat less for the floor-lateral system than for the side-port system previously tested, and pertinent adjustments based on quantitative tests in the model were incorporated in the final prototype design. The laterals adopted are uniform in size throughout their length and are alternately connected at one end only to culverts in the lock walls.

To reduce the velocity of jets entering the lock chamber and to minimize the turbulence created in the lock chamber during the filling operation, it was considered desirable to provide a total area of lock-chamber ports considerably larger than the total lateral area. In the preliminary design, from which the model was built, twelve rectangular ports, 1 ft by 1.83 ft, were provided for each lateral. This size gave a ratio of port area (lock chamber) to lateral area of 1.37. However, experiments in the model indicated that the former area should be increased and that the shape of the ports should be revised somewhat. After tests were made of several sizes, shapes, and spacings of lock-chamber ports, in conjunction with tests on both tapered and uniform laterals and on laterals connected to the culverts at both ends, as well as at alternate ends, a design for lock-chamber ports was adopted providing eleven rectangular ports, 9.5 in. by 4 ft, for each lateral. The ports are equally spaced along the lateral, and well streamlined, to reduce turbulence to a minimum. The arrangement gives a desirable transverse distribution of flow in the lock with a minimum of turbulence and surface disturbance resulting from jet action.

The ratio of port area (in lock chambers) to lateral area is 2.76.

Discharge Ports.—The discharge-port manifolds for the MacArthur Lock were first patterned after a type of manifold developed for Pickwick Lock on the Tennessee River. This type of manifold tapered toward its downstream end to counteract a tendency of the pressure gradient to rise in the direction of flow. Such a tapered discharge manifold was constructed in the model. However, the model tests revealed that the emptying system as originally designed would not be capable of emptying the lock in the desired time and that a considerable improvement in emptying efficiency was necessary. Consequently, it was decided to expand or flare the discharge manifold in the direction of flow.

However, the expansion of the manifold accentuated the previously described tendency of the pressure gradient in the manifold to rise in the direction of flow, and it was necessary to extend the discharge-port piers into the culvert and to shape them so that they would act as flow deflectors or vanes, to obtain an approximately equal distribution of flow through the discharge ports. A discharge manifold and ports, so modified, were installed in the model and, after some minor modifications, were found to operate very satisfactorily.

The final design (see Fig. 8) has four discharge ports for each culvert, providing a ratio of total port-throat area to total culvert area of 1.55.

Results.—The model tests resulted not only in obtaining better lockage and channel-approach conditions by important reductions in surface currents and turbulence, but also in increasing the over-all efficiency of the system. Valve schedules were developed for the recommended design to fill and empty the lock in varying periods to fit contemplated conditions, and anticipated prototype line stresses were determined to be not greater than about 10 tons.

In operation, the prototype has behaved quite like the model. Surface currents in the prototype appear to be remarkably similar to those observed in the model and are extremely minor. The lock coefficients for final design were found in the model to be 0.71 for filling and 0.61 for emptying. Preliminary prototype tests indicate these coefficients to be 0.72 and 0.67, respectively.

The Locks Proposed for the "Chain of Rocks" Reach, Mississippi River

In a canal planned to by-pass the troublesome "Chain of Rocks" reach in the Mississippi River just upstream from St. Louis, Mo., there are 2 parallel locks. The design of the adopted hydraulic systems (see Fig. 9) was determined by local conditions and the results of intensive model experiments.

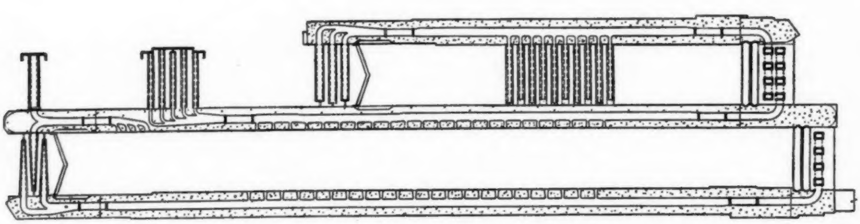


Fig. 9.—Chain of Rocks Locks near St. Louis, Mo.

The normal lift is about 12 ft; the maximum, 21 ft. For the main lock, 110 ft by 1,200 ft, floor intakes, longitudinal wall culverts, and wall ports are used. The floor intakes upstream from the upper sill provide against entrainment of air. Wall ports are used because the water cushion over the floor of the lock will be not less than 25 ft. The ports are streamlined and have rounded entrances and exits to improve flow conditions. They are grouped in the center section of the length of the chamber, the ports in one wall being staggered with respect to those in the opposite wall to permit adjacent jets to pass each other—all according to the results of the model testing. A vertical-lift gate was used as the upper lock gate because of the necessity of passing ice from the 7-mile upper canal, because of the height of the lock walls above the upper sill (58 ft), and because of the favorable elevation of the rock foundation. This type of gate is advantageous in filling the lock. Flow over the gate will be added to, and synchronized with, the flow through the culverts, the filling time for the conditions under a 21-ft lift being thereby shortened 2 min, to a period of 7.5 min. Discharge from the main lock is to be con-

trolled by three valves, one in each of the longitudinal culverts and an additional valve in a stub culvert at the lower end of the intermediate wall. The three exits are required to give an emptying period comparable to the filling period; and the flow discharges into the lower pool through lateral diffusers, as indicated, to provide uniform distribution across the canal prism and to obtain minimum turbulence in the lower approach to the auxiliary lock. Details of a lateral diffuser are shown in Fig. 10.

The auxiliary lock will provide a chamber 110 ft by 600 ft. To minimize surge and longitudinal currents and crosscurrents, it will be filled and emptied by a single wall culvert, fed through a floor intake similar to that provided for the main lock and by a group of floor laterals in the center part of the length of the chamber. The laterals will discharge horizontally through side ports to minimize turbulence further. Discharge will be controlled by a single valve in the side culvert, the flow emptying below the lower miter gates through a lateral diffuser to minimize turbulence and to distribute the flow uniformly across the lower approach.

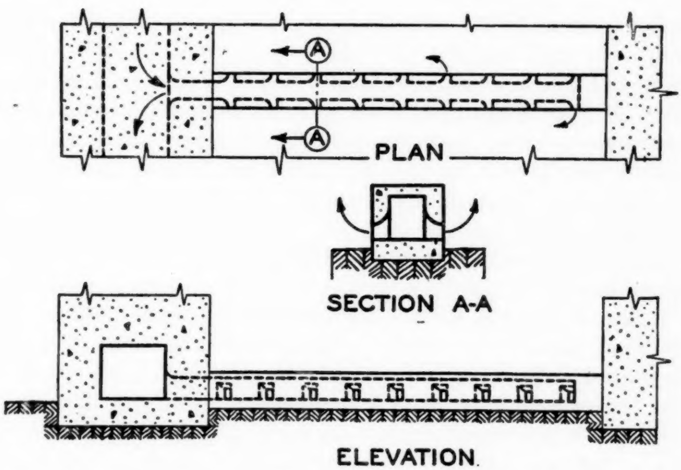


FIG. 10.—LATERAL DIFFUSER, CHAIN OF ROCKS LOCKS, NEAR ST. LOUIS, MO.

Very satisfactory operating conditions were obtained in the model as finally modified. Similar, or even better, operating conditions are confidently expected in the prototype.

PRESENT AND FUTURE USE OF MODELS

Entrance and discharge conditions of the various components of the lock hydraulic system have been greatly improved through the use of lock models. As a result, lock-filling time and lock-emptying time have been reduced in spite of a general increase in the size and the lift of locks, and significant improvement has been obtained in hydraulic conditions within and without the lock chamber. The following data serve to illustrate the use of models in solving some perplexing problems:

(*a*) With a 1:25 scale model, the Bonneville Hydraulic Laboratory has obtained test data on several types of lock-filling and lock-emptying systems to permit the selection of the most satisfactory hydraulic system for a lock 675 ft long and 86 ft wide, with an average head of 85 ft. This lock was designed as a part of the McNary Power and Navigation Project in Umatilla, Ore. The results of the first series of tests indicate that a system incorporating bottom longitudinal culverts is not as satisfactory as one consisting of longitudinal side culverts with fourteen lateral culverts in the lock-chamber floor. Maximum line stresses of 14 tons were developed using the longitudinal system while operating with a 92-ft head and a 4-min valve-opening period as compared to a maximum stress of 2.5 tons with the lateral system.

(*b*) Using a 1:20 scale model, the Waterways Experiment Station at Vicksburg has investigated the feasibility of filling a lock 800 ft long, 75 ft wide, with a minimum depth of 12 ft over the sills, and subject to reverse heads up to a maximum of 18.5 ft, by sector gates. The lock, known as "Algiers Lock," was designed to connect the Intracoastal Waterway from the west with the Mississippi River at New Orleans. Lock-filling tests have been conducted to determine the best distribution of flow between the gate leaves and through the gate recesses, and the optimum gate-travel characteristics. Preliminary test data indicate that it is desirable to pass as large a percentage of the flow as is possible through the gate recesses. It was also determined that, to prevent undue surges in the lock chamber, the initial operation of the gates must be very slow. By use of a variable gate-speed travel to open the gates 5 ft in 5 min, it was found practicable to fill the lock in about 7 min without creating undue turbulence.

(*c*) With a 1:25 scale model, the Hydraulic Laboratory of the Corps of Engineers at Iowa City has investigated the over-all and component parts of proposed hydraulic systems for a main lock 1,200 ft long and 110 ft wide with a difference in pool elevations of 22.6 ft, to develop improvements in the originally selected systems. These locks are planned as a part of a replacement and improvement program on the Ohio River near New Cumberland, Ohio. Longitudinal culverts in the river and middle walls connected to intake manifolds in the sides of the approach walls, to lock-chamber ports in the sides of the walls, and to outlet manifolds discharging into the tail bay below the lower miter gates are proposed for the main lock. The auxiliary lock is filled and emptied by an intake manifold, a longitudinal culvert, and a discharge manifold in the land wall, the longitudinal culvert being connected to laterals with side ports in the chamber floor. Tests on the main lock have shown that modifying the lock-chamber ports so that the jets from the ports strike the floor of the lock chamber at the base of the opposite wall, instead of at the midpoint of the floor, results in a reduction in hawser stresses by an average of 50%. With an assumed 2-min time of culvert-valve operation, satisfactory filling in about 8 min and emptying in about 8.4 min were obtained. It was also found that reducing the inside area of the laterals in the auxiliary lock chamber, in steps from the first port to the end, would result in a more equal distribution of flow during filling operations.

(*d*) In addition to the foregoing model studies, a study is currently being conducted at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota in Minneapolis, by the St. Paul (Minn.) District of the Corps of Engineers in connection with a proposed extension of the Upper Mississippi River canalization project above St. Anthony Falls. The project will incorporate two locks 56 ft wide and 400 ft long. The upper lock will have a 50-ft lift and the lower lock, a short distance downstream, will have a lift of 25 ft. A model scale of 1:22.4 is being used. The purposes of the investigation are to determine the hydraulic characteristics of the Tainter gates (upper lock gates) and chambers in the proposed locks when operated as spillways, to study the filling conditions in the chambers when used for lockages, and to develop a one-culvert, bottom-filling system for the lower lock.

(*e*) The need for extensive generalized information on lock-filling and lock-emptying systems has long been recognized. The St. Paul District has been delegated to make generalized model tests on high-lift locks for the Corps of Engineers. It is contemplated that the program will furnish data on the operating characteristics of locks for heads as high as 100 ft. These data will include the limiting rates of rise versus cushion depth for various types of lock-filling systems, the loss of head for various units of the hydraulic system, methods of minimizing vortices at the intake section in the upper bay, surge and pressure tests on culvert Tainter valves, dimensions and spacing of lock-chamber laterals with side-wall ports, proportionment and expansion of culvert flow in the discharge section, and the formation of waves and surges in the lock chamber. Supplementing the foregoing program, prototype tests are contemplated to obtain stage-time data from filling and emptying operations, head-loss data for computing coefficients for component parts; and, for the over-all systems, data on distribution of flow, turbulence, surges, waves, vortices, and other disturbances, measurements of forces acting on typical craft in lockage, and data on the cushion depths required to obtain satisfactory operating conditions for various rates of filling. It is anticipated that the gathering and analyzing of these data will require several years.

Concluding Remarks

Unless obviously adverse operating conditions can be tolerated, or unless economic studies indicate a cost prohibitively greater than that of a less desirable system, it is considered that the hydraulic system offering the maximum efficiency consistent with economy and safe and relatively quiet lockage should be adopted.

Experience proves that lock operators and navigators will soon forget the economic considerations that may have determined the selection of a lock hydraulic system, but will evaluate the system, and those who designed it, from the manner in which the lock operates.

Lock hydraulic systems should be designed so that they can be operated as planned without creating intolerable lockage conditions in the lock chamber or intolerable mooring conditions in the upper and lower approaches. It is believed to be both possible and practicable to design systems that will provide

fast and safe operation under various pertinent conditions. The side-culvert and the side-port system will fulfil the requirements for safe and fast lock filling where the initial water cushion in the chamber is adequate. On the other hand, for high-lift, shallow-draft locks, where fast operation is required, the bottom-lateral system with side ports offers a solution to the problem. In the event that turbulence and currents in the lower approach, created by lock discharge, need correction to permit safe mooring of vessels not too far from the lower lock gates, a diffuser is recommended. Lock models can be used to solve difficult hydraulic problems efficiently and practically. Much is yet to be done in this field and the continued use of models is strongly recommended.

Acknowledgments

The writer wishes to acknowledge his deep appreciation of the fine spirit of cooperation shown by all who contributed their time and effort to the preparation of this paper. Many of the findings recorded herein were obtained through model and prototype tests at, or under the direction of, personnel connected with the Hydraulic Laboratory of the Corps of Engineers, at Iowa City. Other results were obtained from tests made at other laboratories, such as those at Bonneville and Panama and at the TVA Hydraulic Laboratory, and from experiences of, and information given by, those cooperating with the writer in furnishing basic data for this paper.

STRUCTURAL ELEMENTS FOR EMERGENCY CLOSURES AND UNWATERING OPERATIONS

By C. E. Blee,[7] M. ASCE

Introduction

A review of past and present practice indicates that the provisions with which this paper is concerned, made at navigation locks, fall into one of the following three classifications:

1. Provisions for unwatering the lock and its operating equipment for ordinary routine inspection and maintenance, in which case closure can be made under conditions of balanced head;
2. Provisions for safeguarding, or preventing damage to, lock operating gates and other equipment; or
3. Provisions for making an emergency closure of the lock under an unrestricted flow of water resulting from damage of disastrous proportions to the lock gates.

These classifications are not sharply defined and in some cases facilities may overlap into two or more groups. The extent to which any or all of these provisions are employed at a given location is dependent upon the importance of the waterway, the natural site conditions, or similar governing limitations, although there may be fundamental differences in concept of the provisions necessary for reliable operation.

As may well be imagined, practice has varied widely over the years. During the early development period, lockage facilities were constructed principally for the purpose of connecting large bodies of navigable water of natural origin, such as lakes. The works at Sault Ste. Marie are a notable example of this type of development. The vital importance of this waterway had long been recognized and the wisdom of using every device to assure reliability of operation has never been questioned. At the time of initial development, much thought was evidently given to the problem of devising methods for stopping the unrestricted flow that would result from direct communication of water levels—that is, for emergency operations. This attitude has persisted throughout the development, and over the years several types of structures for accomplishing this purpose have been designed and built. Apparently there has been no occasion to use them for the designed purpose on the United States side; and, on the Canadian side, there has been only one such occasion.

Again in the design of the new Welland Canal connecting Lake Erie and Lake Ontario, also a waterway of great importance, the same problem apparently received much thought. Here, however, consideration led to placing the emphasis on protective measures against damage, rather than to controlling the damage after it had occurred. As a result, reliability in operation is secured

[7] Chf. Engr., TVA, Knoxville, Tenn.

through the use of guard gates or protective fenders, or both, at vulnerable locations in the waterway. As an added feature for maintenance purposes only, wherever necessary an additional set of gates of timber construction is provided to permit the unwatering of the locks.

With the intensive, although somewhat less spectacular, development of rivers as inland waterways (the Ohio, Upper Mississippi, and Tennessee rivers, among others) a fundamentally different attitude has developed. Although attention is still paid to the methods of controlling an unrestricted flow of water through the lock (as reflected by the installation of an emergency dam of the wicket type at Pickwick Landing Dam on the Tennessee River, 206 miles above the mouth), the trend has been toward the use of types of structures suitable only for maintenance purposes, and of the simplest construction. These structures usually take the form of a needle dam or a Poirée dam, although other forms are used to suit special site conditions. They are economical in first cost but are cumbersome and slow in operation. The trend toward their use is the result of a belief, proved by experience, that the probability of accidents leading to the connection of water levels is much less than previously anticipated. Therefore, the emergency gates will seldom be used and the greater expense incurred in operating these somewhat cumbersome types is more than offset by the saving in capital cost.

Structures Intended for Use in the Routine Unwatering of a Lock

Needle Dams.—The needle dam (see Fig. 11) consists of a horizontal girder spanning the lock opening and seated in recesses in the lock walls above pool level, together with approximately vertical beams, or needles, extending between the horizontal girder and a seat on the lock sill. The horizontal girder is a member of substantial proportions the dimensions of which increase rapidly with an increase in width of the lock opening. It has usually been built of structural steel, but efforts to reduce its weight to reasonable limits for handling have led to proposals for the use of structural aluminum. The needles may be of wood, of steel shapes or of built-up members—or a combination of each. The trend is toward larger members of built-up steel sections. The needle dam is suitable for locks of the lesser and moderate widths and the relatively shallow depths. For the larger locks the weight and the size of the members generally preclude its use. It is slow in operation and generally requires treatment with some form of sealing medium, such as cinders, to reduce leakage to a reasonable amount.

Poirée Dams.—Poirée dams comprise a series of trestles, usually in the form of A-frames, spaced at regular intervals across the lock opening (see Fig. 12). These trestles support horizontal beams, or "walers," to which are attached the approximately vertical sheathing elements. The trestles are anchored to the sill by devices embedded in the concrete. This type of dam partakes of all the advantages and disadvantages of the needle dam with the added disadvantage that, in operation, it is even more slow and cumbersome and requires the services of a diver to seat the trestles in the anchor block and attach the framing members below the water level. It is used for locks of greater width where the needle type dam is not practicable.

Horizontal Trusses with Skin Plate.—At the Fort Loudoun Lock, 45 miles downstream from Knoxville on the Tennessee River (which represents the latest design for the TVA system), the sectionalized bulkhead to permit unwatering operations is formed of horizontal trusses with skin plate (see Fig. 13).

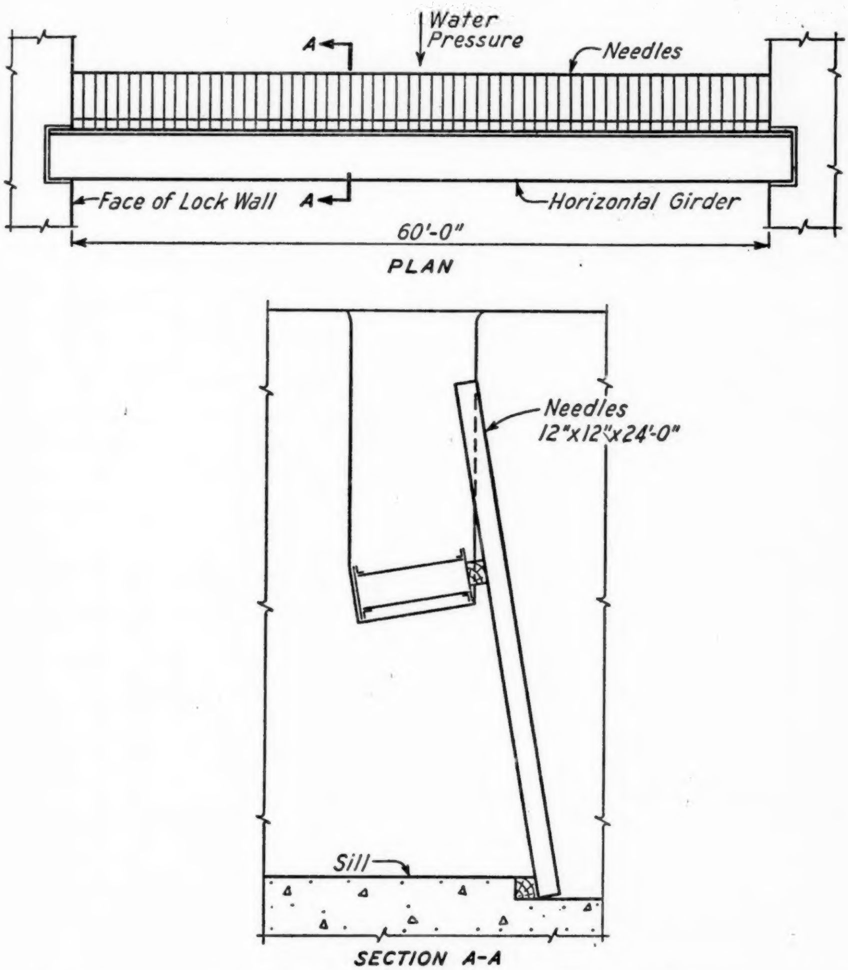


FIG. 11.—UNWATERING A DAM OF THE GIRDER AND NEEDLE TYPE, BASED ON THE INSTALLATION AT WHEELER DAM ON THE TENNESSEE RIVER

Each section consists of two horizontal trusses, an upstream skin plate, and a vertical bracing between trusses. The clear width of the lock is 60 ft and vertical slots in the lock walls form the seats for the trusses. The horizontal trusses have their upstream chord curved in plan and their downstream chord straight. The skin plate is carried on the upstream chord. When in place,

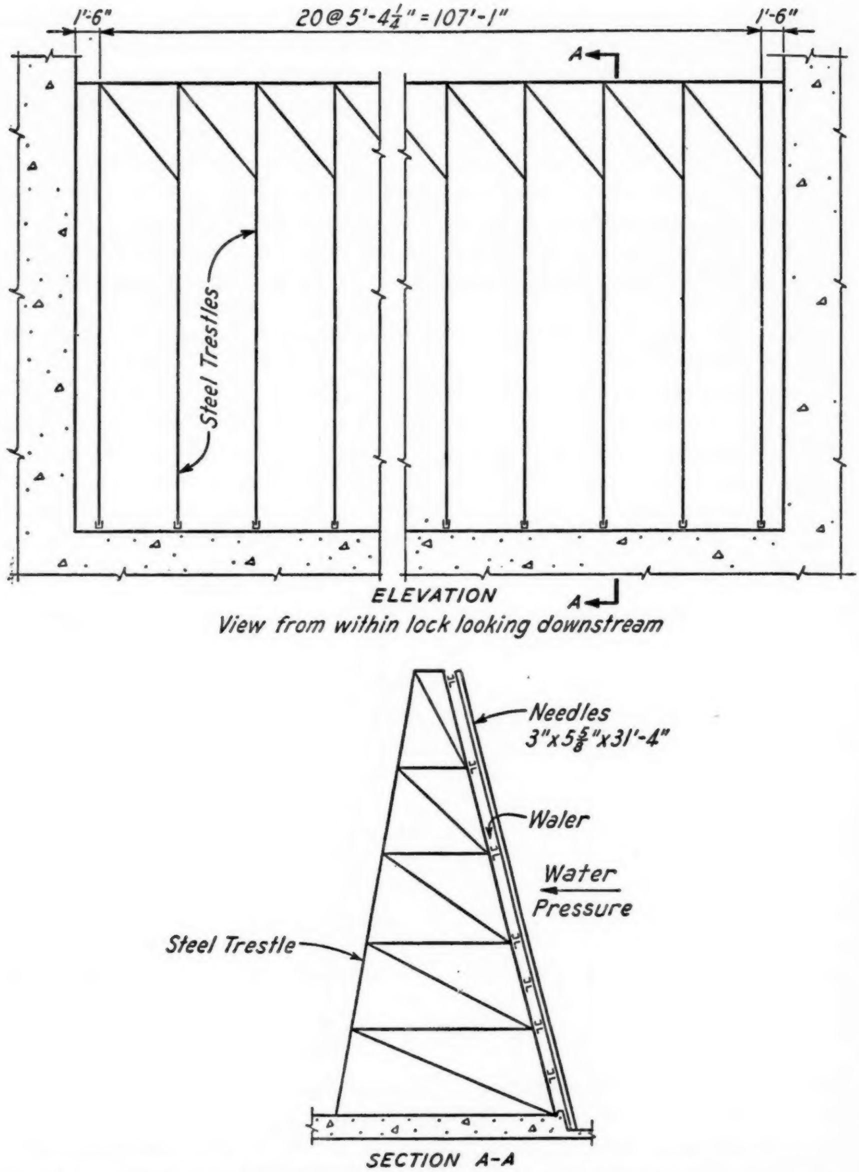


FIG. 12.—UNWATERING A DAM OF THE POIRÉE TYPE, BASED ON THE LOWER INSTALLATION, PICKWICK LANDING DAM ON THE TENNESSEE RIVER

the trusses are spaced close together near the bottom and wider apart near the top, the spacing varying with the head.

The bulkhead sections are stored on land and when needed can be placed by a floating derrick as no permanent derrick is provided. In accordance

with general practice on the TVA system of locks, the same bulkhead can be used at either the upper end or the lower end of the lock and only one bulkhead is provided.

Other Types.—On the inland river waterways the needle and Poirée types of dams are most commonly used for emergency closure of the lock openings, especially where mitering types of service gates are used. At locks where special conditions are encountered, other types of emergency closures have been provided or proposed. Two of these will be mentioned briefly, as follows:

1. A Caisson and Temporary Navigation Gate.—A number of locks on the Ohio River are equipped with service gates of the rolling type—that is,

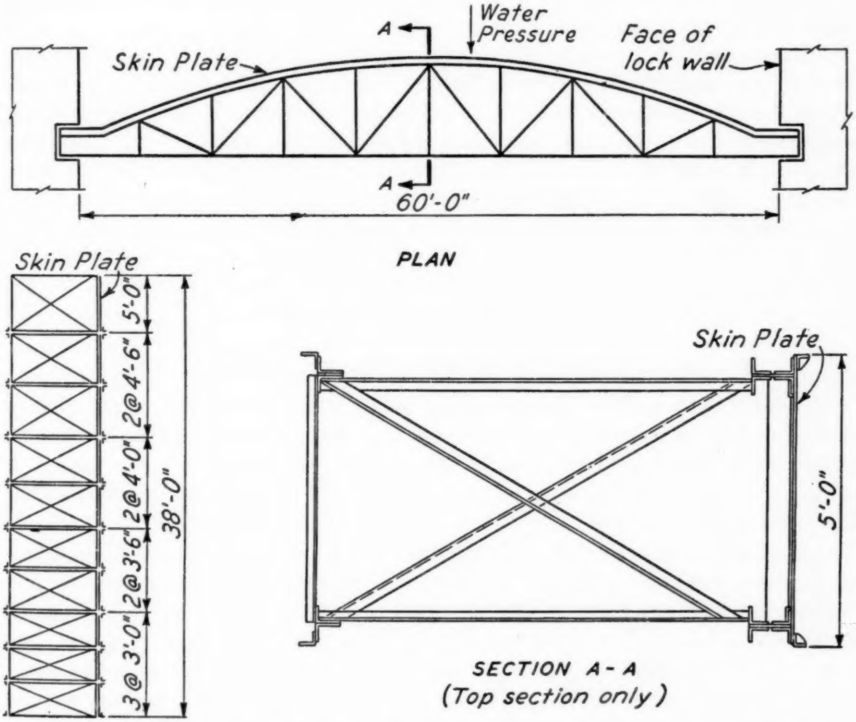


FIG. 13.—UNWATERING A DAM OF THE HORIZONTAL TRUSS AND SKIN PLATE TYPE, BASED ON THE INSTALLATION AT FORT LOUDOUN DAM ON THE TENNESSEE RIVER

each gate is mounted on a large number of wheels and is opened or closed by rolling it on a straight steel track in and out of a recess built into the landward side of the lock. Originally it was planned to repair or renew the track on the floor of the lock chamber by unwatering the lock with Poirée dams at each end. Concern for the safety of the lock walls and the possibility of leakage under those built on piles caused abandonment of this plan to unwater the locks. Instead, a floating coffer or caisson and a temporary navigation gate are provided. The caisson is designed as a buoyant vessel having a broad base and

a vertical access superstructure. When sunk in place over a gate track, it extends halfway across the lock. The temporary gate is used across the other half of the opening. By pumping out the central part of the caisson, access is provided to that part of the gate track covered by the caisson. The other part can be inspected and repaired by reversing the position of the caisson and temporary gate. When the caisson is on the land side of the lock chamber, it bulkheads the gate recess, thus permitting work on the gate. The caisson and the temporary navigation gate are handled by suitable derrick boats. Navigation through one side of the lock is maintained by handling the temporary gate. When the caisson and temporary navigation gate are required at another lock, they are loaded on a barge for transportation. These units are reported to give very satisfactory service and to reduce, materially, the time required for maintenance.

2. A Caisson.—An application proposed for the Kentucky Lock, 22 miles above the mouth of the Tennessee River (but never built), called for a caisson similar to the type employed in closing graving and dry docks. This caisson was designed as a buoyant vessel of trapezoidal cross section which could be floated into position in front of a gate and then sunk to place by the addition of water ballast. It was designed to span the full lock opening and to be supported at its vertical edges by slots in the lock walls, and along its keel by the miter sill of the lock. It was to be provided with the necessary pumps, valves, piping, and watertight compartments for manipulation of the water ballast. This lock as now operated is unwatered with the aid of a Poirée dam.

Structures Intended to Prevent or Minimize Damage

Guard Gates.—Guard gates are provided for the purpose of presenting a second barrier (the first being the service gate) to the direct communication of water levels through the lock. Generally they are placed at the upstream approach to the lock chamber where the hazard is the greatest, although they have also been installed to protect the lower gates at vulnerable locations. Lower guard gates are required to provide complete protection by this method throughout the locking sequence; but it may be argued that once the vessel is within the lock chamber it will be under control and, hence, not likely to damage the lower gates. Guard gates are usually duplicates of the service gates, operated with them in the same manner and in such sequence that they will bear the brunt of collision with a vessel. Neither the service gates nor the guard gates are operable under unbalanced water pressures, but the possibility of an accident serious enough to damage both sets of gates to an extent which would permit any substantial flow of water through the lock is remote. With this in mind, guard gates may be considered effective and adequate protection against the intercommunication of water levels through the lock chamber. Of course, they are expensive; their cost includes not only that of the gates and operating machinery but also that of the necessary extension of the heavy gate thrust block of the lock walls.

Fenders.—Fenders have been built in many forms. They are placed ahead of the gates with the idea of absorbing the shock of collision with a vessel and

arresting its motion before it can cause damage to the gate. The more primitive form consists of nothing more than a chain rigidly attached to the lock walls, hanging just above the water surface when in working position, and dropped to the sill to permit the passage of vessels. Improvements have been made from time to time to increase their resistance to a blow and to facilitate their handling. There are inherent limitations to their use in their lack of resiliency and snubbing action; and, although they do provide a measure of protection, their use on locks of modern construction has largely been discontinued. On the other hand, the idea of the fender has been developed to a high degree in designing the locks of the Welland Canal. Here the fender comprises a 3.5-in. (diameter) wire rope, extending across the lock opening and attached on each wall to snubbing devices which provide a dampening action against the motion of the vessel in cases of collision. The rope is supported on a light bridge structure conforming to the general design of a rolling-lift bascule bridge. At one of the lock-wall faces the rope is separated and provided with matching rope sockets through which a retractable pin is inserted to make the rope continuous when in working position in front of the gates. In case of collision the supporting structure is destroyed, but its replacement amounts to only a minor item. A fender of this type represents a substantial expense. Undoubtedly, it provides a large measure of protection.

Protective measures such as those described are expensive but they are apparently effective in preventing serious damage, judging from the lack of evidence to the contrary. They undoubtedly contribute to a greater sense of security and confidence on the part of both lock operators and navigators. In inverse ratio their existence may lead, in some cases, to a lessening of caution, with unfortunate results to the protective equipment itself, if not to the main operating equipment. Their use will result in slowing the time of transit of a vessel through the lock.

Structures Intended for Use in Closing Off Unrestricted Flow

Emergency Dam of the Bridge Type.—The dam in this design consists of a swing bridge with center of rotation on the lock wall. Under this bridge are mounted girders that rotate in vertical planes about a horizontal axis close to the bottom chords of the bridge trusses. These vertical girders support wickets of some form. In operation the bridge is rotated into position across the lock and seated so that the bottom chord system spans the lock walls. The vertical girders are then rotated to a bearing against the lock sill and the wickets placed. The reactions for the water load are distributed between the lock sill and the bottom chord system of the bridge. This type is suitable for use across openings of moderate width and presumably would function as intended. However, few, if any, of those built have been operated under the design conditions. It is cumbersome in operation and requires considerable space for mounting and storing on the lock wall. Notable examples are the early dams at Sault Ste. Marie.

Emergency Dam of the Lift Type.—This dam consists of bulkhead sections, built as trusses with skin plates, or as box girders, which span the lock en-

trance and, when placed one upon another, form a continuous dam from the sill to the water surface. Rollers are mounted on the ends of the bulkhead and the rollers bear against tracks embedded in slots in the side walls of the lock. The bulkheads are thus, in effect, sections of wheeled gates and can be operated under unbalanced water pressures (see Fig. 14). The bulkheads are handled by derricks mounted on the lock walls. This dam is less complicated in design than the bridge type and less cumbersome in operation. Storage facilities for the girders and mounting of the derricks, however, do require substantial space

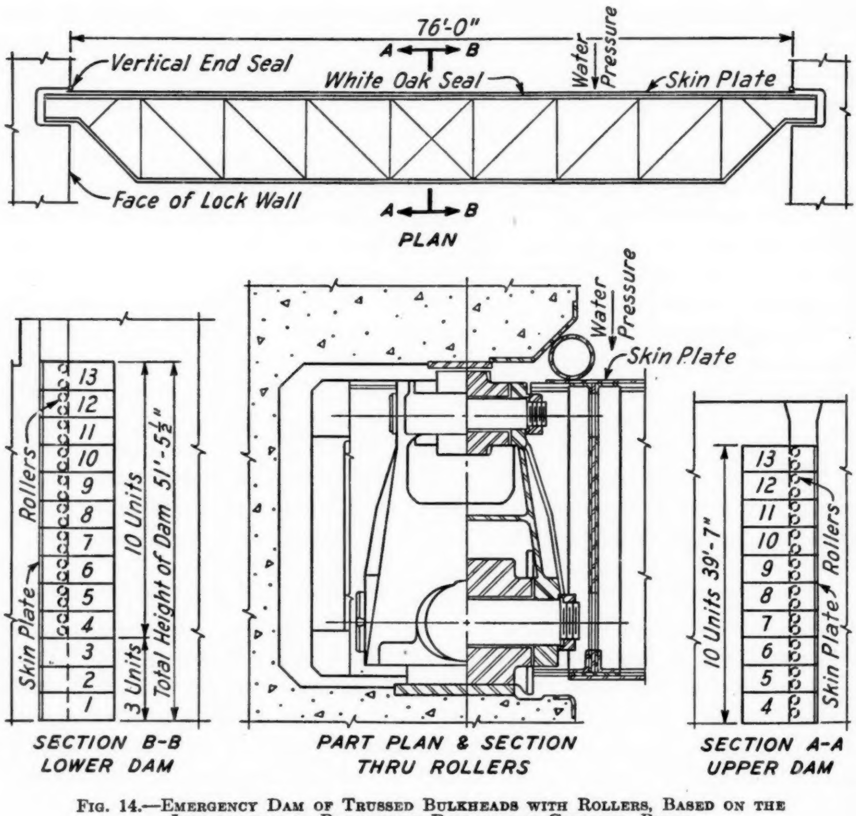


FIG. 14.—EMERGENCY DAM OF TRUSSED BULKHEADS WITH ROLLERS, BASED ON THE INSTALLATION AT BONNEVILLE DAM ON THE COLUMBIA RIVER

allowances on the lock walls. Notable examples are the later dams at Sault Ste. Marie and the closures at Bonneville Dam on the Columbia River.

Emergency Dam of the Wicket Type.—This dam consists of a series of wickets stored in slots in the lock sill. When raised, it supports sectionalized gates designed to operate under unbalanced water pressures. In the raised position the wicket forms a modified **A**-frame (see Fig. 15). In operation the wickets are raised into position by a pull from the lines of the handling derricks exerted through chains permanently attached to the wickets. The derricks also serve

to handle and lower the gates. This type is suitable for the operation of locks up to the maximum width, but of rather shallow depths, such as occur on the waterways developed for barge traffic. The use of portable derricks, stored with the gates in recesses of the lock walls, provides clear working spaces

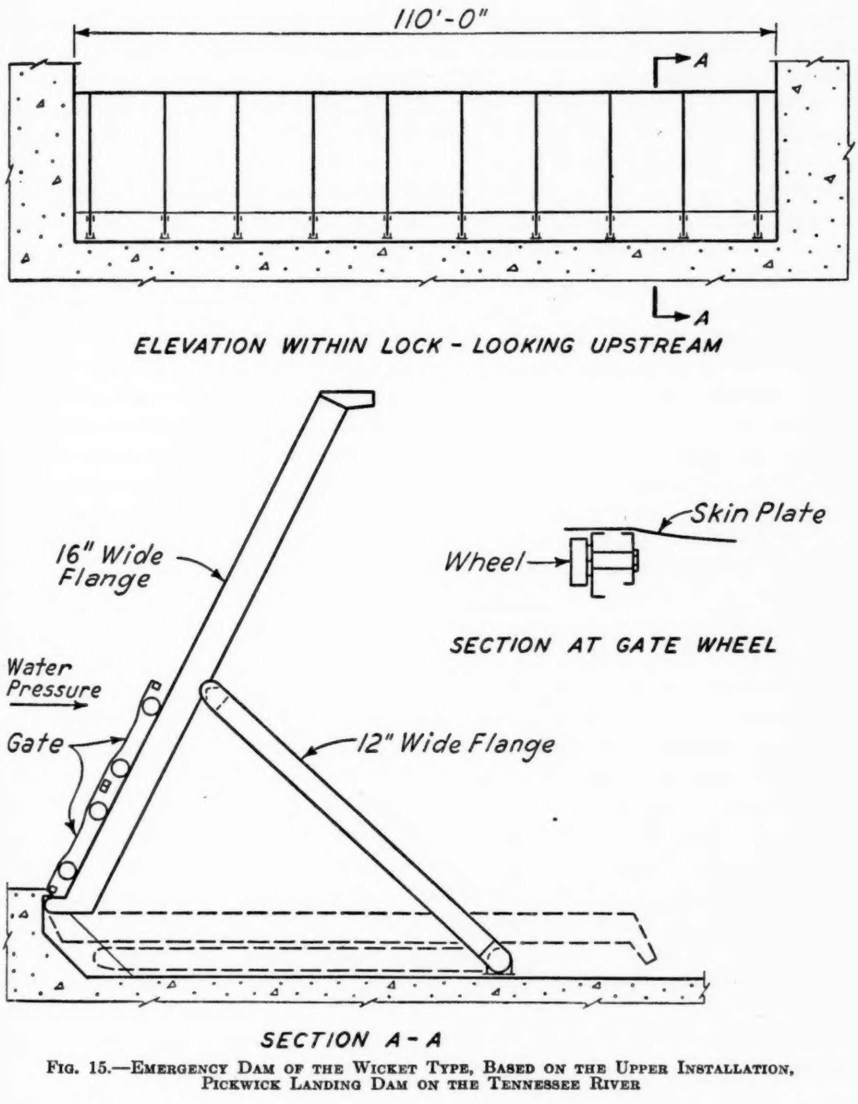


FIG. 15.—EMERGENCY DAM OF THE WICKET TYPE, BASED ON THE UPPER INSTALLATION, PICKWICK LANDING DAM ON THE TENNESSEE RIVER

on the lock walls and presents a much cleaner appearance. The component parts of this dam are not large, and operation is less cumbersome than for the other types. The wickets are continually immersed in the water when stored and are therefore subjected to severe corrosive action without practicable

methods of repair and maintenance. The slots are subject to silting. The degree to which operation is affected is dependent on the progress of these adverse conditions. Care is necessary also in returning the wickets to the storage slots after they have been used to prevent silt or gravel deposits from causing an incompletely lowered wicket to project into the channel clearance. Examples are the emergency dams at Pickwick Landing Dam on the Tennessee River and at the Montgomery Island Locks on the Ohio River, 32 miles downstream from Pittsburgh.

Another type of structure that has been used with success on locks of comparatively narrow width is the adaptation of the radial gate for use as a service lock gate. In this adaption the gate is installed with the axis of the trunnions in a vertical plane near the faces of the lock walls. The gate rotates around this axis from its storage position in a recess in the lock wall to its working position mitered against its matching gate in somewhat the same manner as the conventional miter gate. Because of the circular contour of the face of the gate, concentric with the axis of rotation, there are no significant unbalanced moments of the water loads against the rotation of the gate, thus permitting operation under any conditions of water pressure, either balanced or unbalanced. When both upper and lower lock gates are of this design, there is adequate assurance that the one may act as an emergency gate in case of serious damage to the other. As far as is known, no gates of this type have been used as gates or dams for emergency use only; but it does offer a possibility of a service gate that could act as an emergency gate to close against free flow. It is cited as a possible solution to some of the problems which have long confronted the designers of navigation locks.

Other types of emergency dams have been built; but, since they have appeared only in isolated instances and since the designs probably will not be repeated, they are not included in this paper.

Except where a long approach canal permits the location of the emergency dam far enough upstream to be outside the zone affected by an accident in the immediate vicinity of the lock gates, they are installed on the upper sills immediately upstream from the gates. In this position it is quite possible that the wreckage of the gates or of the vessel will interfere with the operation of the dams. Since this is the region in which accidents are most likely to occur, there arises the question of whether or not even the most mechanically reliable and efficient dam will accomplish its purpose under these circumstances. This consideration and the infrequency of their need have combined to create a belief that, except under special conditions, provision of emergency dams capable of closing under an unrestricted flow of water is not justified. In this connection it may be noted that in the case of the 9 highlift locks on the Tennessee River there has never been an occasion that would have required the use of emergency equipment to close the lock against a condition of unbalanced head.

Summary

In any discussion of this subject, there emerges the ever recurrent question of the control of flow through the lock chamber in the event of direct inter-

communication of water levels. During the early development of large-scale lock construction, it apparently was considered to be of far greater importance than it is today. At present, it occupies a prominent place principally in connection with additions to installations where previous practice has hardened into a set pattern, or where the great importance of the waterway justifies the installation of this rather expensive equipment. This long-standing problem was reviewed comprehensively, in 1945, by the late Isaac De Young,[8] M. ASCE.

In more recent times the trend has been toward reliance on precautionary and protective measures on waterways of great importance. On waterways where interruption of traffic would not cause a crippling loss of transportation facilities, the trend is toward the more simple and less expensive types of closure suitable only for ordinary maintenance work. This practice seems fully justified, and may well have been encouraged, by the rarity of accidents of crippling proportions with modern locking facilities.

[8] "A Plan for a Movable Dam," by Isaac De Young, *Transactions*, ASCE, Vol. 111, 1946, p. 253.

LOCK SIZES FOR INLAND WATERWAYS

By Ralph L. Bloor,[9] M. ASCE

Synopsis

Data on the location, size, and date of construction of navigation locks on the inland waterways of the United States are presented, together with historical information serving to establish, in some important cases, the reasons for the sizes selected in the past. Conclusions regarding past practice are drawn. Factors governing the selection of lock sizes are enumerated and data are presented regarding the size of existing barges and towboats, and the make-up of tows. Trends in barge and towboat sizes and towing practices are discussed. Standard sizes for locks which are considered to fit best all the controlling elements are suggested.

Introduction

The size of a navigation lock to be constructed on a stream or waterway is an important subject worthy of careful study in each individual case. It is important because the structure is expensive to build, in the first place, and is extremely difficult to alter later. The size has a major effect on the success of the waterway. It may be so small as to stifle development of the traffic it is built for; or it may be so unnecessarily large as to place fixed and operating charges beyond the economic limit. It is hoped that the following observations will serve to promote a more uniform and complete consideration of the subject.

Scope

The paper presents data on the horizontal dimensions of the chamber between lock walls and between the upper and lower gates of navigation locks. The facts set forth, and the comments and conclusions derived from the facts are concerned primarily with navigation locks on inland waterways of the United States where the cargoes are carried in barges which are assembled with a power towboat into a tow. They apply only incidentally and partly to navigation locks in harbors or canals, which essentially serve ocean ships or lake ships. The lock widths are true distances between lock walls except for 2 in. or 3 in. which often are added to allow for construction inaccuracies. The lock lengths stated are the so-called usable lengths—that is, the lengths that can be utilized by tows occupying full widths without being struck by the movement of the lock gates. For the most common type of lock with mitering lock gates, the usable length is measured from the downstream side of the upper miter sill where it joins the lock wall to the upstream point of the lower gate when it is in the open position. The stated lengths may be accurate only to within 1 ft or 2 ft because of the common practice of spacing the pintle points of upper and lower mitering gates an even number of feet apart.

[9] Chf., Structures Branch, Corps of Engrs., U. S. Dept. of the Army, Washington, D. C.

EXISTING LOCKS

Fig. 16 is a partial map of the United States on which are marked all the waterways that contain operating navigation locks. There are about 360 locks, a few of which are older than 100 years. There are other streams, of course, navigated by other methods, and because of the small scale of the map it has not been practicable to designate the location of individual locks. Some idea can be obtained, however, of the geographical distribution of the canalized waterways and some estimation can be made of one question germane to this paper: Is there a need for considering the characteristics of connecting waterways when studying lock dimensions on any one stream?

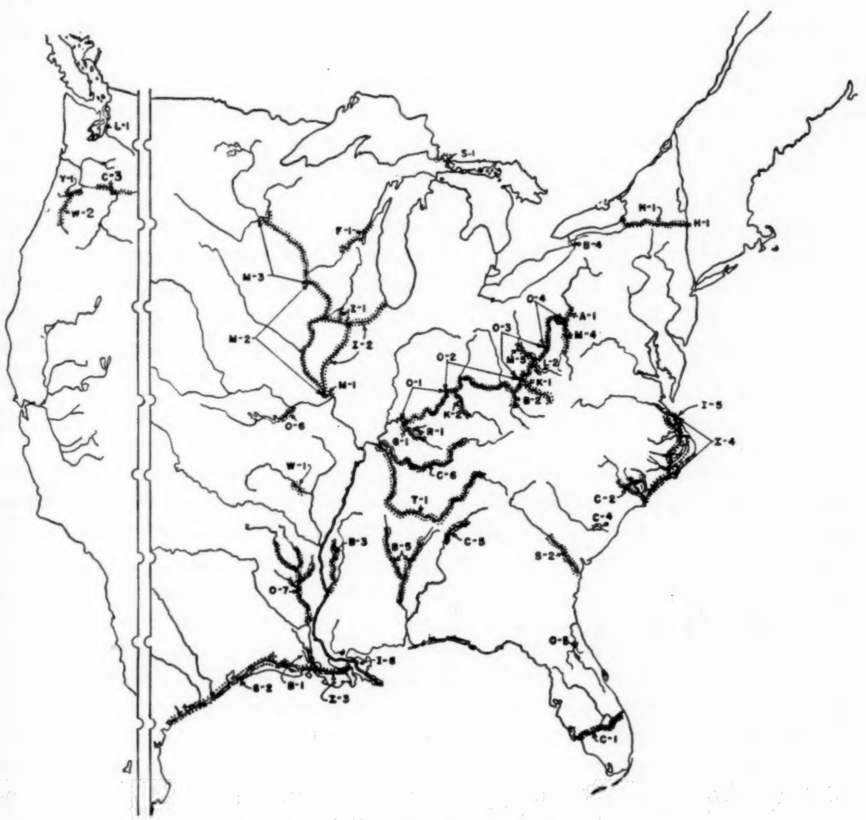


FIG. 16.—CANALIZED WATERWAYS OF THE UNITED STATES
(LETTERS AND NUMERALS REFER TO TABLE 1)

The Mississippi and Ohio rivers and their tributaries form the most extensive system of canalized waterways. This system connects at its southern end with the Intracoastal Waterway along the north and west shores of the Gulf of Mexico, and the latter waterway in turn joins the Alabama, the Tom-

bigbee, and the Warrior rivers in Alabama, and the Coosa River in Alabama and Georgia as tributaries to the same system. Other canalized streams on the east and west coasts, including the Erie Canal in New York, are quite isolated from the Mississippi-Ohio system at present. Relatively little new construction would be necessary to effect a continuous system along the east coast which would include the Erie Canal, and there is no insurmountable obstacle to connecting the east coast system to the Mississippi-Ohio system at one or more points. It appears, therefore, that in making long-range plans—and all lock-size planning should be at long range—consideration must be given to facilities on actual or possible connecting streams if the project being studied is in the Mississippi River Basin or to the east or south of it. The west coast developments will doubtless remain isolated and the only attention that needs to be given the eastern streams is an appraisal of the experience developed from them during the many years they have been navigated.

Table 1 furnishes certain information on all the operating navigation locks in the United States. There are 364 of them exclusive of parallel or auxiliary locks, and the oldest was built in 1836. Of course, many of them have supplanted older locks on the same streams and it is known that navigation locks were built in the United States well before the beginning of the nineteenth century. Fig. 17 shows the date of construction of all the existing locks plotted against the area of the lock chamber. It serves to indicate the variations in lock sizes constructed in the past and to illustrate the usual trend in American structures from the small sizes of many years ago to the largest sizes in recent years. The lock size on the Ohio River is a notable exception. (Not shown in Fig. 17 are 57 New York State Barge Canal locks, 45 ft by 300 ft, completed in 1918.)

History

It is interesting and instructive to review some of the history of lock construction with special attention to the reasons for adopting certain lock sizes. Some of the reasons, of course, can be expected to be different from those which exist today, but one of the engineering problems remains much the same—to adopt a size that will be adequate (but not extravagantly large) to serve the traffic that will develop during the expected life of the structure.

The New York State Barge Canal is one of the oldest canalized waterways. Action to build it started in 1768 when freight costs from New York, N. Y., to points along the route were $75 per ton to $100 per ton and New York City was losing fur trade to Canadian cities and streams. Under these circumstances the problem that overshadowed everything else was to create some kind of waterway from the Hudson River in New York to the Great Lakes. It was needed quickly and its considerable length and use of private capital discouraged any long-range planning. By 1796 locks 10 ft wide and 70 ft long, permitting a 2-ft draft, had been built and freight costs dropped to $32 per ton one way and $16 per ton the other way. By 1825 the State of New York had completed 84 locks between Albany and Buffalo 15 ft wide and 90 ft long for a waterway with a 4-ft draft. This number had been reduced by 1862 to 72 locks 18 ft wide and 110 ft long. Some of these locks were lengthened in 1897-

TABLE 1.—Navigation Locks of the United States

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft): Width (4)	Dimensions (Ft): Length (5)	Dimensions (Ft): Lift (6)
Group A-1. Pennsylvania and New York; Depth, 9 Ft					
Allegheny River	2	1934	56.0	360.0	11.0
	3	1934	56.0	360.0	13.5
	4	1927	56.0	360.0	10.5
	5	1927	56.0	360.0	11.8
	6	1928	56.0	360.0	12.2
	7	1930	56.0	360.0	13.1
	8	1931	56.0	360.0	17.9
	9	1938	56.0	360.0	22.0
Group B-1. Bayou Teche, La.; Depth, 8 Ft					
Keystone Lock	..	1913	36.0	160.0	1–8
Group B-2. West Virginia and Kentucky; Depth, 6 Ft					
Big Sandy River	1	1905	54.6	160.0	13.5
	2	1905	54.8	160.0	12.6
	3	1897	51.9	158.0	10.6
Levisa Fork	1	1910	55.0	160.0	11.0
Tug Fork	3	1910	55.0	160.0	12.0
Group B-3. Mississippi; Depth, 4.5 Ft					
Big Sunflower River	1	1918	36.0	160.0	...
Group B-4. Buffalo, N. Y.; Depth, 21 Ft					
Black Rock Channel	..	1914	68.0	625.0	5.0
Group B-5. Alabama; Depth, 9 Ft					
Tombigbee River	1	1908	52.0	281.9	12.0
	2	1914	52.0	286.0	9.0
	3	1914	52.0	286.0	10.0
Warrior River	4	1908	52.0	285.6	10.0
	5	1908	52.0	285.6	10.0
	6	1908	52.0	285.6	10.0
	7	1903	52.0	284.7	10.0
	8	1903	51.7	284.7	10.0
	9	1903	52.0	284.7	11.0
	13	1905	52.0	285.5	11.0
Black Warrior River; Tuscaloosa Lock	..	1939	95.0	460.0	30.0
	14	1910	52.0	282.1	14.0
	15	1910	52.0	282.1	14.0
	16	1915	52.0	285.5	21.0
	17	1915	52.0	285.5	72.0
Group C-1. Florida;[a] Depth, 6 Ft to 12 Ft					
New St. Lucie Lock[b]	..	1943	50.0	250.0	14.0
Old Lock No. 2[b,c]	2	1925	30.0	130.0	14.0
Moore Haven Lock	..	1935	50.0	250.0	2.5
Ortona Lock	..	1937	50.0	250.0	11.5

TABLE 1.—(*Continued*)

Waterways	Lock No.	Year opened	Dimensions (Ft)		
			Width	Length	Lift
(1)	(2)	(3)	(4)	(5)	(6)
Group C-2. North Carolina; Depth, 8 Ft to 30 Ft					
Cape Fear River	1	1916	40.0	200.0	11.0
	2	1917	40.0	200.0	9.0
	3	1935	40.0	300.0	9.0
Group C-3. Columbia River, Oregon; Depth, 27 Ft					
Bonneville Lock	..	1938	76.0	500.0	63.8
Big Eddy Lock	..	1941	45.0	265.0	45.0
Fivemile Lock	..	1915	45.0	265.0	10.1
Tenmile Lock	..	1915	50.0	265.0	6.0
Celilo Lock	..	1915	45.0	265.0	6.0
Group C-4. South Carolina; Depth, 4 Ft					
Congaree River	..	1904	55.0	170.0	10.0
Group C-5. Georgia and Alabama; Depth, 4 Ft					
Coosa River	4	1914	52.0	280.0	8.0
	3	1890	40.0	176.0	8.0
	2	1890	40.0	176.0	5.51
	1	1890	40.0	176.0	5.33
Lock at Mayos Bar	..	1913	40.0	176.0	12.0
Group C-6. Kentucky and Tennessee; 6 Ft[d]					
Cumberland River	F	1923	52.0	280.0	11.5
	E	1922	52.0	280.0	10.1
	D	1916	52.0	280.0	10.9
	C	1918	52.0	280.0	12.3
	B	1916	52.0	280.0	11.8
	A	1904	52.0	280.0	13.3
	1	1904	52.0	280.0	5.5
	2	1907	52.0	280.0	9.0
	3	1908	52.0	280.0	11.1
	4	1909	52.0	280.0	13.3
	5	1909	52.0	280.0	10.7
	6	1910	52.0	280.0	13.7
	7	1910	52.0	280.0	10.3
	8	1924	52.0	280.0	12.7
	21	1911	52.0	280.0	19.5
Group F-1. Fox River, Wisconsin; Depth, 4 Ft to 6 Ft					
De Pere Lock	..	1936	36.0	146.0	8.9
Rapide Croche Lock	..	1934	36.0	146.0	8.3
Kaukauna Lock:					
Little	..	1938	36.0	146.0	7.2
Fifth	..	1898	35.6	144.0	9.1
Fourth	..	1879	36.6	144.1	10.2
Third	..	1879	36.6	144.0	10.2
Second	..	1903	35.0	144.0	9.6
First	..	1883	35.1	144.4	11.0
Little Chute Locks:					
Lower Combined	..	1879	35.4	146.5	10.9
Upper Combined	..	1879	36.3	144.1	10.6
Second	..	1881	35.0	144.2	13.8
Cedars Lock	..	1888	35.0	144.0	9.8

TABLE 1.—(*Continued*)

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft) Width (4)	Length (5)	Lift (6)
Group F-1.—(*Continued*)					
Appleton Locks:					
Fourth	..	1907	35.0	144.0	7.6
Third	..	1900	35.0	144.0	8.7
Second	..	1901	35.1	144.6	9.6
First	..	1884	35.0	144.7	10.0
Menasha Lock	..	1899	35.4	144.0	8.5
Eureka Lock	..	1876	35.0	148.6	2.0
Berlin Lock	..	1878	34.8	148.6	1.3
White River Lock	..	1878	34.5	148.5	1.9
Princeton Lock	..	1878	34.9	148.4	1.5
Grand River Lock	..	1878	34.7	148.3	1.4
Montello Lock	..	1901	35.3	137.0	3.8
Governor Bend Lock	..	1931	35.0	137.0	3.8
Fort Winnabago Lock	..	1936	34.7	137.0	6.4
Portage Lock	..	1928	35.2	146.0	2.3
Group G-1. Kentucky; Depth, 5 Ft					
Green River	1	1840	35.5	139.0	11.8
	2	1895	36.0	141.0	14.3
	3	1836	35.8	137.5	17.0
	4	1839	35.8	138.0	16.4
	5	1934	56.0	360.0	15.2
	6	1905	36.0	145.0	9.2
Barren River	1	1934	56.0	360.0	15.2
Group G-2. Gulf Intercoastal Waterway;[e] Depth, 12 Ft					
Harvey Lock	..	1934	75.0	425.0	0–20
Plaquemine Lock	..	1909	55.0	260.0	0–38
Vermilion Lock	..	1933	56.0	1,182.0	0–5
Group H-1. Hudson River, Troy, N. Y.; Depth, 27 Ft					
Federal Lock	..	1917	44.4	492.5	17.3
Group I-1. Illinois; Depth, 7 Ft					
Canal between Illinois River and Mississippi River	1	1907	35.0	143.08	6.7
	2	1907	35.0	143.08	9.0
	3	1907	35.0	143.08	9.0
	4	1907	35.0	143.08	9.0
	5	1907	35.0	143.08	8.0
	6	1907	35.0	143.08	10.0
	7	1907	35.0	143.08	8.0
	8	1907	35.0	149.75	8.0
	9	1907	35.0	149.75	8.0
	10	1907	35.0	149.75	9.0
	11	1907	35.0	149.75	9.0
	12	1907	35.0	149.75	8.0
	13	1907	35.0	149.75	10.0
	14	1907	35.0	149.75	10.0
	15	1907	35.0	149.75	10.0
	16	1907	35.0	149.75	11.0
	17	1907	35.0	149.75	10.0
	18	1907	35.0	149.75	9.0
	19	1907	35.0	149.75	10.0
	20	1907	35.0	149.75	11.0
	21	1907	35.0	149.75	11.0

TABLE 1.—(*Continued*)

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft) Width (4)	Length (5)	Lift (6)
Group I-1.—(*Continued*)					
Guard Lock	..	1907	35.0	149.0	1.8
Rock River Lock	..	1907	35.0	143.08	9.2
Canal between Illinois River and Mississippi River	22	1907	35.0	143.08	9.0
	23	1907	35.0	143.08	11.0
	24	1907	35.0	143.08	11.0
	25	1907	35.0	143.08	8.0
	26	1907	35.0	143.08	9.0
	27	1907	35.0	143.08	8.0
	28	1907	35.0	143.08	8.0
	29	1907	35.0	143.08	11.0
	30	1907	35.0	149.00	0.0
	31	1907	35.0	145.42	6.0
	32	1907	35.0	143.08	7.7
Group I-2. Illinois Waterway; Depth, 9 Ft					
La Grange Lock	..	1939	110.0	600.0	10.00
Peoria Lock	..	1939	110.0	600.0	11.00
Starved Rock Lock	..	1933	110.0	600.0	18.70
Marseilles Lock	..	1933	110.0	600.0	23.95
Dresden Island Lock	..	1933	110.0	600.0	21.75
Brandon Road Lock	..	1933	110.0	600.0	34.00
Lockport Lock	..	1933	110.0	600.0	40.00
Lockport Lock	..	1933	22.0	130.0	40.00
Blue Island Lock	..	1933	50.0	360.0	0
Group I-3. Inland Waterway, Franklin, La., to Mermentau River, Louisiana; Depth, 5 Ft					
Hanson Lock	..	1907	26.5	937.0	1–3
Schooner Bayou Lock	..	1913	36.0	300.0	1–2
Group I-4. Inland Waterway, Norfolk, Va., to Beaufort Inlet, N. C.; Depth, 12 Ft					
Tidal Guard Lock	..	1932	75.0	600.0	2.7
Group I-5. Inland Waterway, Norfolk, Va., to the Sounds of North Carolina; Depth, 10 Ft					
Deep Creek Lock, Va.	..	1940	52.0	300.0	12.0
South Mills Lock, N. C.	..	1941	52.0	300.0	12.0
Group I-6. Inland Waterway, New Orleans, La.; Depth, 30 Ft					
Inner Harbor Lock	..	....	75.0	640.0	0–20
Group K-1. Kanawha River, W. Va.; Depth, 9 Ft					
Twin Locks:					
Winfield	..	1937	56.0	360.0	28.0
Marmet	..	1934	56.0	360.0	24.0
London	..	1934	56.0	360.0	24.0

TABLE 1.—(*Continued*)

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft) Width (4)	Length (5)	Lift (6)
Group K-2. Kentucky River, Kentucky; Depth, 6 Ft					
Lock Nos.	1	1882	38.0	145.0	18.23
	2	1882	38.0	145.0	13.94
	3	1882	38.0	145.0	13.16
	4	1882	38.0	145.0	13.22
	5	1886	38.0	145.0	15.00
	6	1894	52.0	147.0	13.95
	7	1897	52.0	147.0	15.30
	8	1900	52.0	146.0	18.66
	9	1903	52.0	148.0	17.34
	10	1904	52.0	148.0	17.00
	11	1906	52.0	148.0	18.00
	12	1910	52.0	148.0	16.00
	13	1915	52.0	148.0	18.00
	14	1917	52.0	148.0	18.00
Group L-1. Lake Washington Ship Canal, Seattle, Wash.; Depth, 30 Ft					
Large Lock	..	1916	80.0	760.0	6.5–25
Small Lock[e]	..	1916	30.0	123.0	6.5–25
Group L-2. Little Kanawha River, West Virginia; Depth, 4 Ft					
Lock Nos.	1	1874	23.0	125.0	6.4
	2	1874	23.0	125.0	11.3
	3	1874	23.0	125.0	11.0
	4	1874	23.0	125.0	11.9
	5	1891	25.5	125.0	12.4
Group M-1. Upper Mississippi River; Depth, 9 Ft					
Lock Nos.	26	1938	110.0	600.0	23.0
	..	1938	110.0	360.0	23.0
	25	1939	110.0	600.0	15.0
	24	1940	110.0	600.0	15.0
Group M-2. Mississippi River; Depth, 9 Ft					
Upper River	22	1938	110.0	600.0	10.2
	21	1938	110.0	600.0	10.5
	20	1936	110.0	600.0	10.0
	19	1913	110.0	358.0	38.2
	18	1937	110.0	600.0	9.8
	17	1939	110.0	600.0	8.0
	16	1937	110.0	600.0	9.0
	15	1934	110.0	600.0	16.0
	..	1934	110.0	360.0	16.0
Le Claire Lock (Canal)	..	1922	80.0	320.0	11.0
Upper River	14	1939	110.0	600.0	11.0
	13	1938	110.0	600.0	11.0
	12	1938	110.0	600.0	9.0
	11	1937	110.0	600.0	11.0
Group M-3. Upper Mississippi River; Depth, 9 Ft					
Lock Nos.	10	1936	110.0	600.0	8.0
	9	1938	110.0	600.0	9.0
	8	1937	110.0	600.0	11.0
	7	1937	110.0	600.0	8.0
	6	1936	110.0	600.0	6.5

TABLE 1.—(*Continued*)

Waterways	Lock No.	Year opened	Dimensions (Ft)		
			Width	Length	Lift
(1)	(2)	(3)	(4)	(5)	(6)
Group M-3.—(*Continued*)					
Lock Nos.	5-A	1936	110.0	600.0	5.5
	5	1935	110.0	600.0	9.0
	4	1935	110.0	600.0	7.0
	3	1938	110.0	600.0	8.0
	2	1930	110.0	600.0	12.2
	..	1930	110.0	500.0	12.2
	1	1917	56.0	400.0	35.9
	..	1917	56.0	400.0	35.9
Group M-4. West Virginia and Pennsylvania; Depth, 7 Ft to 9 Ft					
Monongahela River Lock Nos.	2	1905	56.0	362.0	8.7
	..	1905	56.0	362.0	8.7
	3	1907	56.0	360.0	8.2
	..	1907	56.0	720.0	8.2
	4	1932	56.0	360.0	10.6
	..	1932	56.0	720.0	10.6
	5	1909	56.0	360.0	12.4
	6	1916	56.0	360.0	13.1
	7	1925	56.0	360.0	15.0
	8	1925	56.0	360.0	15.0
	10	1903	56.0	182.0	10.2
	11	1903	56.0	182.0	10.7
	12	1903	56.0	182.0	10.7
	13	1904	56.0	182.0	10.7
	14	1903	56.0	182.0	10.7
	15	1903	56.0	182.0	10.7
Group M-5. Muskingum River, Ohio; Depth, 4 Ft to 5 Ft					
Lock Nos.	1	1890	55.8	179.0	4.29
	2	1840	35.8	160.0	10.76
	3	1840	35.8	160.0	14.19
	4	1840	35.8	160.0	9.32
	5	1840	35.8	160.0	10.76
	6	1840	35.8	160.0	12.33
	7	1840	35.8	160.0	10.10
	8	1840	35.9	160.0	10.96
	9	1891	35.8	160.0	11.01
	10	1840	35.4	156.5	15.43
	..	1840	35.4	158.5	15.43
	11	1910	36.0	160.0	11.75
Group N-1. New York State Barge Canal; Depth, 14 Ft					
Erie Canal	2	1918	45.0	300.0	33.55
	3	1918	45.0	300.0	34.50
	4	1918	45.0	300.0	34.50
	5	1918	45.0	300.0	33.25
	6	1918	45.0	300.0	33.00
	7	1918	45.0	300.0	27.00
	8	1918	45.0	300.0	14.00
	9	1918	45.0	300.0	15.00
	10	1918	45.0	300.0	15.00
	11	1918	45.0	300.0	12.00
	12	1918	45.0	300.0	11.00
	13	1918	45.0	300.0	8.00
	14	1918	45.0	300.0	8.00
	15	1918	45.0	300.0	8.00
	16	1918	45.0	300.0	20.50
	17	1918	45.0	300.0	40.50
	18	1918	45.0	300.0	20.00
	19	1918	45.0	300.0	21.00

TABLE 1.—(*Continued*)

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft): Width (4)	Dimensions (Ft): Length (5)	Dimensions (Ft): Lift (6)
Group N-1.—(*Continued*)					
Utica Harbor Lock	..	1918	45.0	300.0	7.00
Erie Canal	20	1918	45.0	300.0	16.00
	21	1918	45.0	300.0	25.00
	22	1918	45.0	300.0	25.00
	23	1918	45.0	300.0	6.90
Oswego Canal	1	1918	45.0	300.0	10.20
	2	1918	45.0	300.0	17.80
	3	1918	45.0	300.0	27.00
	5	1918	45.0	300.0	18.00
	6	1918	45.0	300.0	20.00
	7	1918	45.0	300.0	14.50
	8	1918	45.0	300.0	11.10
Group N-1. New York State Barge Canal; Depth, 12 Ft					
Erie Canal	24	1918	45.0	300.0	11.00
	25	1918	45.0	300.0	6.00
	26	1918	45.0	300.0	6.00
	27	1918	45.0	300.0	12.50
	28-A	1918	45.0	300.0	19.50
	28-B	1918	45.0	300.0	12.00
	29	1918	45.0	300.0	16.00
	30	1918	45.0	300.0	16.40
	32	1918	45.0	300.0	25.10
	33	1918	45.0	300.0	25.10
	34	1918	45.0	300.0	24.60
	35	1918	45.0	300.0	24.50
Cayuga and Seneca Canal	1	1918	45.0	300.0	7.50
	2	1918	45.0	300.0	24.50
	3	1918	45.0	300.0	24.50
	4	1918	45.0	300.0	14.50
Champlain Canal	1	1918	45.0	300.0	14.30
	2	1918	45.0	300.0	18.50
	3	1918	45.0	300.0	19.50
	4	1918	45.0	300.0	16.00
	5	1918	45.0	300.0	19.00
	6	1918	45.0	300.0	16.50
	7	1918	45.0	300.0	10.00
	8	1918	45.0	300.0	11.00
	9	1918	45.0	300.0	16.00
	11	1918	45.0	300.0	12.00
	12	1918	45.0	300.0	15.50
Group O-1. Ohio River; Depth, 9 Ft					
Lock Nos.	53	1929	110.0	600.0	13.4
	52	1928	110.0	600.0	12.0
	51	1929	110.0	600.0	8.0
	50	1928	110.0	600.0	10.0
	49	1928	110.0	600.0	11.0
	48	1922	110.0	600.0	7.0
	47	1928	110.0	600.0	9.0
	46	1928	110.0	600.0	11.0
	45	1927	110.0	600.0	9.0
	44	1925	110.0	600.0	7.0
	43	1921	110.0	600.0	9.0
	41	1921	110.0	600.0	37.0
	..	1929	56.0	360.0	37.0

TABLE 1.—(*Continued*)

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft) Width (4)	Length (5)	Lift (6)
Group O-2. Ohio River; Depth, 9 Ft					
Lock Nos.	39	1921	110.0	600.0	6.0
	38	1924	110.0	600.0	7.3
	37	1911	110.0	600.0	7.8
	36	1925	110.0	600.0	7.9
	35	1919	110.0	600.0	6.4
	34	1925	110.0	600.0	5.6
	33	1921	110.0	600.0	7.0
	32	1926	110.0	600.0	7.5
	31	1919	110.0	600.0	7.5
	30	1923	110.0	600.0	7.5
	29	1916	110.0	600.0	8.0
Group O-3. Ohio; Depth, 9 Ft					
Ohio River	28	1915	110.0	600.0	7.1
	27	1923	110.0	600.0	6.4
Gallipolis Locks	..	1937	110.0	600.0	26.0
	..	1937	110.0	360.0	26.0
Ohio River	23	1921	110.0	600.0	5.6
	22	1918	110.0	600.0	7.8
	21	1919	110.0	600.0	5.6
	20	1917	110.0	600.0	7.5
	19	1916	110.0	600.0	7.7
	18	1910	110.0	600.0	6.2
	17	1918	110.0	600.0	8.2
	16	1917	110.0	600.0	7.8
	15	1916	110.0	600.0	7.8
	14	1917	110.0	600.0	8.3
Group O-4. Ohio; Depth, 9 Ft					
Ohio River	13	1911	110.0	600.0	7.3
	12	1917	110.0	600.0	8.4
	11	1911	110.0	600.0	7.3
	10	1915	110.0	600.0	8.4
	9	1914	110.0	600.0	7.4
	8	1911	110.0	600.0	6.4
	7	1914	110.0	600.0	6.9
Montgomery Island Locks	..	1936	110.0	600.0	19.4
	..	1936	56.0	360.0	19.4
Dashields Locks	..	1929	110.0	600.0	10.0
	..	1929	56.0	360.0	10.0
Ensworth Locks	..	1936	110.0	600.0	18.0
	..	1936	56.0	360.0	18.0
Group O-5. Oklawaha River, Florida; Depth, 6 Ft					
Moss Bluff Lock	..	1925	30.0	125.0	11.5
Group O-6. Osage River, Missouri; Depth, 3 Ft					
Lock	1	1906	42.0	220.0	16.0

TABLE 1.—(*Continued*)

Waterways (1)	Lock No. (2)	Year opened (3)	Dimensions (Ft) Width (4)	Length (5)	Lift (6)
Group O-7. Ouachita and Black Rivers, in Arkansas; Depth, 6.5 Ft					
Lock Nos.	2	1918	55.0	268.0	14.40
	3	1919	55.0	268.0	14.80
	4	1915	55.0	268.0	8.90
	5	1924	55.0	268.0	6.90
	6	1913	55.0	268.0	9.70
	8	1912	55.0	268.0	13.50
Group R-1. Rough River; Kentucky; Depth, 4 Ft					
Lock	1	1896	27.0	123.0	9.9
Group S-1. St. Marys River, Michigan; Depth, 21 Ft to 25 Ft					
MacArthur Lock	..	1943	80.0	800.0	21.7
Poe Lock	..	1896	95.0	800.0	21.7
Davis Lock	..	1914	80.0	1,350.0	21.7
Sabin Lock	..	1919	80.0	1,350.0	21.7
Group S-2. Savannah River, Georgia; Depth, 6 Ft					
Lock	..	1937	56.0	360.0	15.0
Group T-1. Tennessee River in Tennessee, Alabama, and Kentucky; Depth, 9 Ft					
Kentucky Lock	..	1942	110.0	600.0	56.0
Pickwick Landing	..	1937	110.0	600.0	55.0
Tennessee River	1	1926	60.0	300.0	2.3
Wilson Lock:					
Upper Lift	..	1927	60.0	292.0	91.7
Lower Lift	..	1927	60.0	300.0	91.7
Wheeler Lock	..	1934	60.0	360.0	48.0
Guntersville Lock	..	1937	60.0	360.0	39.0
Hales Bar	..	1913	60.0	265.0	34.0
Chickamauga Lock	..	1937	60.0	360.0	51.0
Watts Bar Lock	..	1941	60.0	360.0	58.0
Fort Loudoun Lock	..	1943	60.0	360.0	72.0
Group W-1. White River, Arkansas; Depth, 4 Ft					
Batesville Lock	1	1904	36.0	147.0	15.6
Earnharts Lock	2	1905	36.0	147.0	12.8
Walls Ferry Lock	3	1908	36.0	147.0	14.5
Group W-2. Willamette River, Oregon; Depth, 6 Ft					
Lock Nos.	1	1873	37.0	175.0	22.5
	2	1873	37.0	175.0	8.7
	3	1873	37.0	175.0	10.9
	4	1873	37.0	175.0	8.1
Group Y-1. Yamhill River, Oregon; Depth, 4 Ft					
Lafayette Lock	..	1900	40.0	175.0	16.0

[a] Caloosahatchee River, Lake Okeechobee, Florida. [b] At Stuart, Fla. [c] Auxiliary. [d] Project depth at numbered locks, 4.6 ft. [e] From Apalachee Bay, Florida, to the Mexico-United States border.

1898 to 208 ft or 230 ft. Still later the number of locks was reduced to 57 and the lock dimensions were (and still are) 45 ft by 300 ft. Thus, in a little more than 100 years the locks were enlarged four times. Of course, in a waterway such as this the locks form a much more minor part of the total system than is the case where large natural waterways are used to a greater extent. For this reason there is more justification than usual for increasing lock sizes only after a demand for a larger channel has developed, rather than for constructing locks to a size believed desirable for future anticipated requirements.

The Monongahela River development is one of the oldest and still one of the busiest waterways. Originally it was important because it was the first navigable stream west of the Allegheny Mountains (in the eastern United States) which was reached by the national road (present highway, United States route 40). The first traffic was by flatboats and rafts of westward moving settlers and, for the protection of this traffic, there was a law prohibiting the construction of dams with lifts greater than 4.5 ft. There are records of armed resistance to the construction of locks and dams. However, 4 locks and dams were built between 1840 and 1844 with the locks 50 ft wide and 158 ft long. This is a rather large size for such an early date and it may have been influenced by the fact that many flatboats were built on the Monongahela River, floated downstream as far as New Orleans with various cargoes, and then broken up and the lumber and timbers sold for the construction of other things. It may well have been that planking about 25 ft long was generally available in the Monongahela forests of that day and consequently the greatest width of barges and flatboats was fixed at that length. At any rate, a barge width of from 26 ft to 27 ft is widely used today and is generally known as the Monongahela River size. All the locks that have been built on the river vary

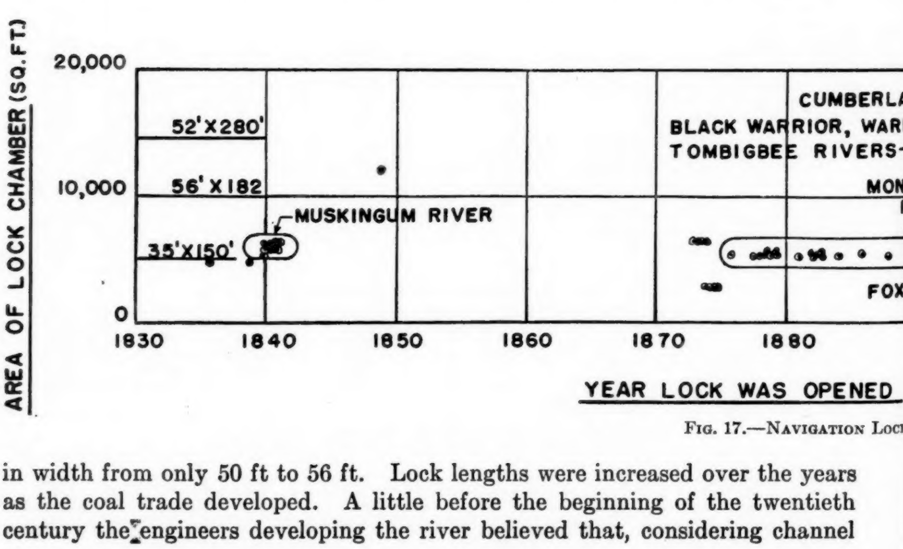
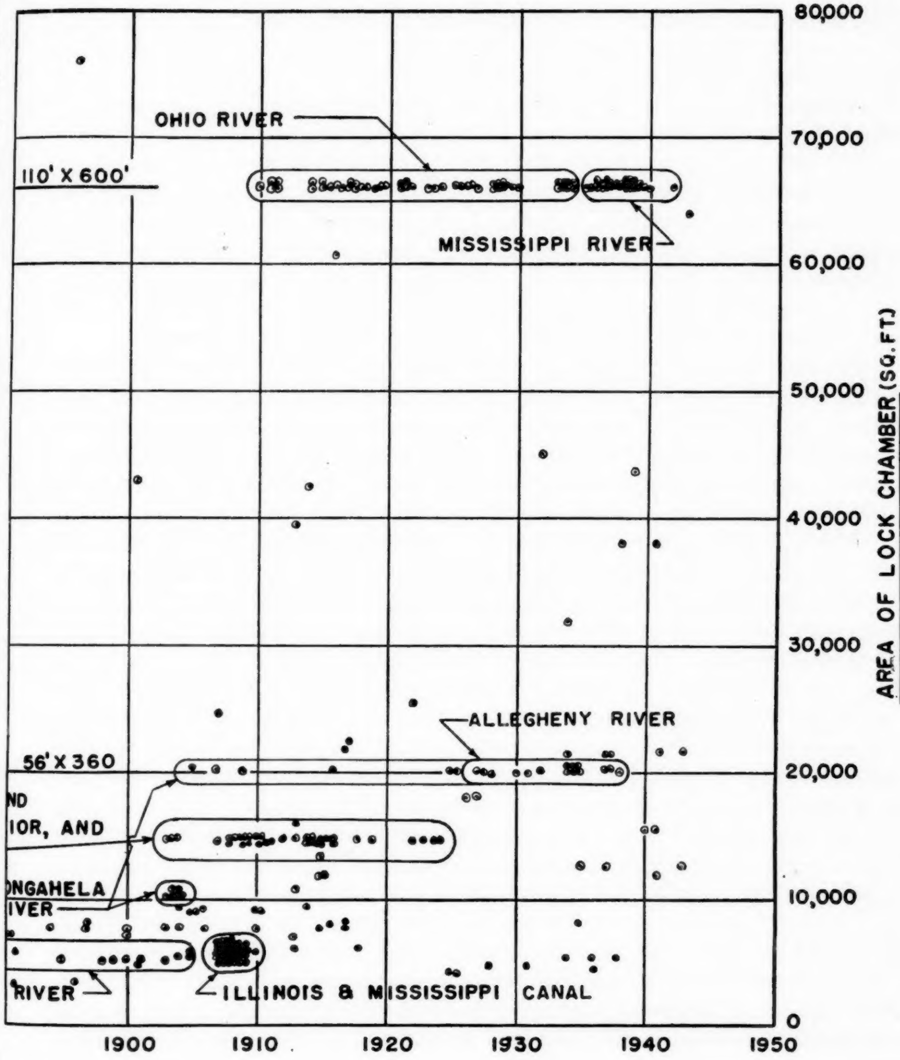


FIG. 17.—NAVIGATION LOCKS

in width from only 50 ft to 56 ft. Lock lengths were increased over the years as the coal trade developed. A little before the beginning of the twentieth century the engineers developing the river believed that, considering channel

limitations, experience indicated that a fleet carrying 3,000 tons on an 8-ft draft was about the limit of safety and that disposition in three coal boats, from 26 ft to 27 ft wide by 175 ft long, was most convenient. Such a tow



TO NAVIGATION

OF THE UNITED STATES

needed a lock 350 ft long. As a result, a lock size of 56 ft by 360 ft was fixed. This size has been used on many other streams and has been varied on the Monongahela only by doubling its length to 720 ft where traffic is densest.

Similar reasoning was applied in planning locks on the Cumberland, Warrior, and other rivers. Lock sizes of 52 ft by 280 ft were adopted on these two streams to accommodate three barges 25 ft by 130 ft and a towboat 150 ft long. On the Tennessee River there was another influencing factor. The State of Alabama in 1830 had built lock and canal systems at Muscle Shoals, the worst restriction to open-river navigation on that stream. This early work fell into disuse and when the complete canalization of the river was later undertaken a lock width of 60 ft was adopted because the original canals at Muscle Shoals were 60 ft wide.

The records show repeatedly that many lock sizes were selected to accommodate the largest boats and tows using the stream under open-river conditions. There may be some criticism of this basis inasmuch as a canalized stream should be reliably able to accommodate larger tows than could be safely operated under open-river hazards. The records also reveal, in some cases, that the engineers did not overlook this latter consideration but did have to neglect it for the purpose of curtailing costs.

The Ohio River has a long active navigation history. It was navigated for more than 100 years before it was canalized, and the tonnages it has carried have increased steadily during the 60 or more years since the first lock and dam were built. By 1870, or thereabouts, the possibility of securing a reliable 6-ft channel throughout its length of nearly 1,000 miles by the continuation of open-channel methods seemed remote. On the other hand, the navigation interests at that time were reluctant to accept the alternative of canalization. The traffic was predominately coal, moving downstream, and the operation involved large fleets of barges operating during the spring rises in the river. The type of dam developed for the Ohio River was a compromise between two needs—(1) dams to provide pools for navigating during low water, but (2) dams which could be lowered to the bed of the river to permit boats to pass over them during higher stages. The lock size selected was also affected by this need for compromise. In 1874 the engineers reported that locks should be large enough to pass a coal fleet of ten barges 24 ft by 130 ft, one fuel flat 22 ft by 100 ft, and one steamboat 48 ft by 230 ft. Two barges abreast would require locks 50 ft wide, but this would be too narrow for some of the steamboats then in use. If the barges were four abreast, a width of 100 ft would be needed, and it was doubted that a mitering lock gate could be designed for this width. Therefore, a lock 75 ft wide seemed desirable and the corresponding length needed was about 600 ft. To the coal carriers, accustomed to open-river operations, a lock 75 ft wide seemed hopelessly restrictive. In an effort to reach a satisfactory solution the engineers designed a rolling type of lock gate which they considered usable on locks wider then 100 ft, and in 1878 recommended a lock size 110 ft wide by 600 ft long. Thus was originated the largest commonly used lock size in the United States. By reference to Fig. 17 it will be seen how much larger it was than any other lock built up to that time and, furthermore, how the size has continued to be the largest used since then, including the quite recent and very important Upper Mississippi River development. It may be said that the engineers planned well for the future on

the Ohio River; but it must be admitted that they were influenced by navigation interests which did not want any locks at all.

Conclusions on the Basis of Experience

In trying to learn from the foregoing experience, there is no need to be critical of the decisions made in the past. There are many diverse influences that can be imagined to have existed but that cannot be confirmed in any specific case without much more research than is worthwhile. It is quite sufficient simply to judge the results produced and to use the conclusions so drawn as a basis for guiding the planning of the future.

Experience indicates that the most successful waterways are those in which liberal lock sizes were adopted at a relatively early date. It is not true, of course, that large lock sizes will insure an active waterway; but it does appear that relatively large locks are necessary to attract and hold traffic which otherwise would revert to other modes of transportation.

No record has been uncovered of a lock being replaced because it was worn out. On the other hand, many have been replaced or augmented because they were too small. Locks are structures capable of long life. There are several locks that have been in service more than 100 years. They have been repaired and supplied with new gates and sills, but the main structure is the original structure. There are many locks approaching 50 years of age. Some on the Monongahela River have operated at very nearly their full capacity for many years, and in addition were subjected to freezing weather, acid waters, and countless heavy blows from floating plant; yet, if they were large enough, they could be rehabilitated at relatively small cost to serve many more years. Therefore, on the basis of experience, it seems proper to conclude that a lock size which is determined by the size of boats or tows then commonly using the waterway is almost certain to be too small before it is worn out.

Experience suggests that a similar attitude should be adopted regarding (*a*) the relation of lock size to width and alinement and (*b*) the character of adjacent navigable channels. It has been relatively simple to enlarge channels in response to traffic demands, but the enlargement of locks in this manner is so much less feasible that the usual procedure appears to have been the construction of new locks. Furthermore, the canalization of a stream reduces navigation hazards as compared to open-river conditions, and this fact in itself supports the use of larger tows.

Finally, regarding experience, it appears that since the needs of the United States, and its transportation patterns, have changed with growth, many locks have fallen into disuse and in these instances the smaller locks represent smaller economic losses. Furthermore, other changes have rendered locks obsolete before they became inoperable because of age. Progress in engineering design and materials has made higher and higher lifts acceptable so that a reduction in the number of lock and dam structures with consequent saving in lockage time has been possible. Just below Pittsburgh several of the original Ohio River dams were replaced for the principal reason that a different type of dam was needed, one that would maintain more stable pool levels in the important

harbor area. These and similar developments in the past make it desirable to consider carefully whether a proposed lock will have the long usage of which it, as a structure, is capable.

Factors Governing Lock-Size Selections

Costs.—Certainly one of the most important elements in selecting a lock size is cost. The writer has attempted to establish some rough, general relationship between increases in lock sizes and increases in costs but found it impossible; it must be concluded that the relation between size and cost varies widely and is completely dependent on the local conditions at the site. Under favorable conditions an increase of a certain percentage in the lock length or in the lock width may be accompanied by an increase in the cost of only one fourth as much. At other sites, the percentage increase in the cost might even be more than the percentage increase in the length or the width. The reasons for these variations are rather obvious, but it is important in making comparative estimates that none of them be overlooked. Lock gates and operating machinery become heavier, per unit of gate area, as lock widths are increased; and, furthermore, filling and emptying valves become larger with increases in either length or width if the same operating time is to be maintained. The larger locks also require more expensive treatment of lock-filling ports and discharge points because larger volumes of water are moved in a limited operating period. Locks situated in open rivers require guide walls along the riverbank upstream and downstream from the lock, and generally these must be as long as the lock chamber itself. Therefore, if a lock chamber is lengthened, the two chamber walls and also both the upper guide wall and the lower guide wall should be lengthened by an equal amount. For the best operating conditions, the landward lock wall and the guide walls must be in one straight line, and in many natural streams it will be difficult to find a reach straight enough to accommodate such a structure with satisfactory alinement of approach in each direction. The only alternative is to set the lock and approaches back into a convex shore, and the accompanying excavation often causes the cost of the longer locks to increase disproportionately. Additional width also has a similar effect if (as is often the case) nearly the full width of the natural channel is needed for the dam. Foundation conditions, of course, are likely to be more nearly at an optimum for a small lock than for a large one. Extra length and width in cofferdams, where they are required, entail extra costs for larger locks. In spite of the aforementioned factors, the basic requirement of only two lock gates (regardless of length of lock) and only two lock walls (regardless of width) tends in many cases to keep the cost increases accompanying the lock-size increases within acceptable limits.

Economics.—The practice of making economic studies in the selection of a lock size is now rather generally accepted. Details of the methods of evaluating the delay time of boats and tows using a smaller lock as compared to using a larger lock are not unusual and need not be treated in this paper. However, it is important that basic assumptions be correct to insure an acceptable result. The assumed traffic density should correspond to the computed amount of

traffic which will develop, and usually this will involve the waiting time at locks (especially of the smaller sizes). In addition to time lost in smaller locks due to the necessity of breaking tows into several lockages, there will be time lost waiting for a turn to lock. Even at locks that operate below their capacity, delays in waiting time occur when several tows arrive at the lock at the same time. Another factor that is sometimes overlooked is that a tow of a given size can be brought safely into a larger lock more quickly than into a smaller lock. Therefore, there is some delay involved in a smaller lock even for tows that would not fill a larger size. Time studies should recognize the fact that breaking a tow transversely is a simpler and faster operation than breaking it longitudinally.

Even after all tangible factors are considered in an economic study of lock-size requirements, some thought should be given to other cogent circumstances which are practically impossible to evaluate. Smaller locks increase the wear and tear on floating plant and are more often damaged and put out of service by floating plant. Shippers hesitate to use a service that may be delayed even when shipping costs are materially lower. For these and similar reasons, therefore, economic studies of tangible costs and values should not be the sole basis for lock-size determination.

Boats and Barges.—The size of boats and barges that will use a lock is a vital consideration in selecting the size of the lock. Figs. 18, 19, and 20 show the number of barges, of various widths, in operation on several of the main waterways of the United States in 1945. These illustrate the great preponderance of barges from 26 ft to 27 ft wide (all of which will hereafter be designated as 26-ft barges). More than 70% of the barges of this width are 175 ft long and carry from 750 tons to 1,250 tons each, with drafts ranging from 8 ft to 11 ft. A few have been built as long as 210 ft, but most of the remaining 20% or so are less than 175 ft. The next most common barge is from 34 ft to 35 ft wide (which hereafter will be called a 35-ft barge). About one half of these vessels are 195 ft long and carry 1,500 tons on a 9-ft draft. A few of this width have been built longer than 200 ft but most of the others are less than 195 ft.

Fig. 18 shows quite clearly the effect of long established lock sizes on barge construction. The 26-ft barge width is designed to fit the 56-ft lock widths on the Monongahela River, the Kanawha River, and other Ohio River tributaries. These same barges also fit the 110-ft-wide locks on the Ohio River as do also the 35-ft barges. Other barge widths on the Ohio River are notable by their scarcity. On the Mississippi River system (Fig. 19) there are locks 110 ft wide from St. Louis to Minneapolis and downstream from St. Louis navigation is by open river. Although most of the barges on the Mississippi River are either 26 ft wide or 35 ft wide, there are more barges 40 ft wide (or wider) than there are on the Ohio River. Fig. 20 for the Louisiana-Texas Intracoastal Waterway shows less tendency toward standardization in barge widths, doubtless because much of the traffic need not proceed through locks.

An examination of the size of barges constructed during the 8-year period from 1937 to 1945 as compared to the total in operation (see Fig. 21) shows

that on the Ohio and Mississippi rivers a slightly smaller percentage of new barges are 26 ft wide than is indicated by the total in operation. On the Ohio River alone the percentage of new barges in this width has risen as may be noted by comparing the data in Table 2. Barge construction in the 35-ft widths has risen sharply in the same 8-year period. There are fewer new barges in most other widths. Both the total number, and the increased widths in new barges (40 ft, 48 ft, and 50 ft), are too small to be significant. It seems quite

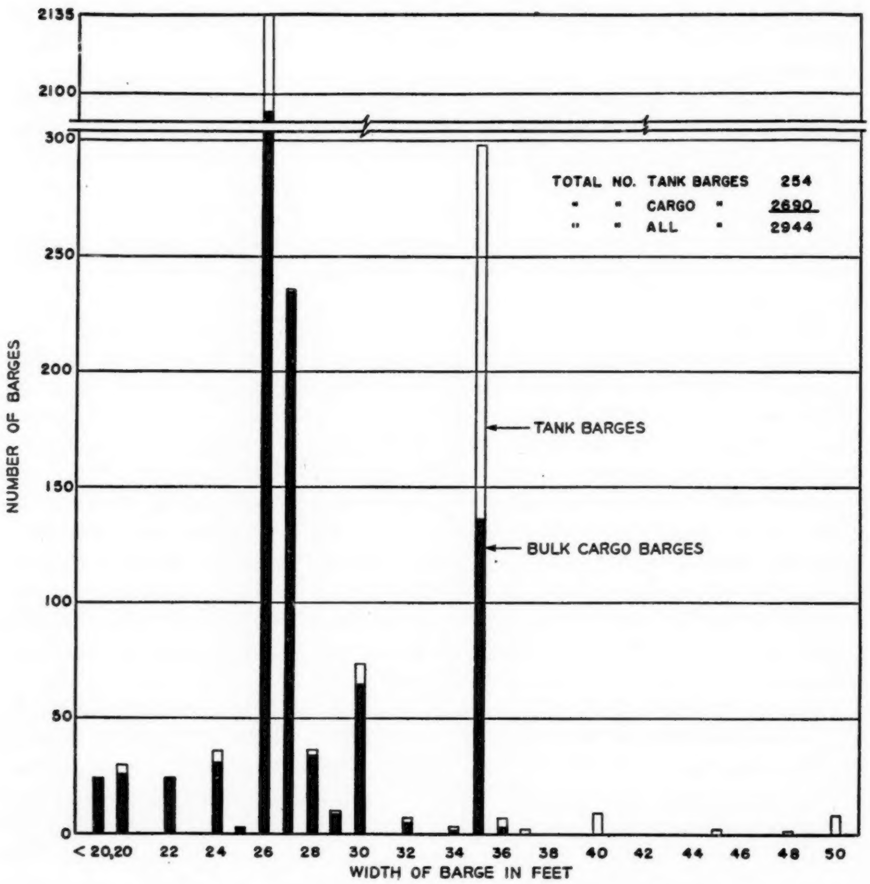


FIG. 18.—WIDTH OF BARGES ON THE OHIO RIVER

evident that there is a definite tendency to standardize the 26-ft-wide and the 35-ft-wide barge sizes, and that even the open-river reach, more than 1,000 miles long, in the Lower Mississippi River, has not caused any substantial adoption of wider barges.

There are a few factors in the construction and operation of barges which might be mentioned as having an effect on optimum barge size. In general, the cost of a barge, per ton of cargo, reduces as the size increases. The ends

of the barge represent more expensive construction than the midship sections and, as a result, there is an advantage in building long barges. Large barges have several disadvantages from an operational point of view. It is difficult

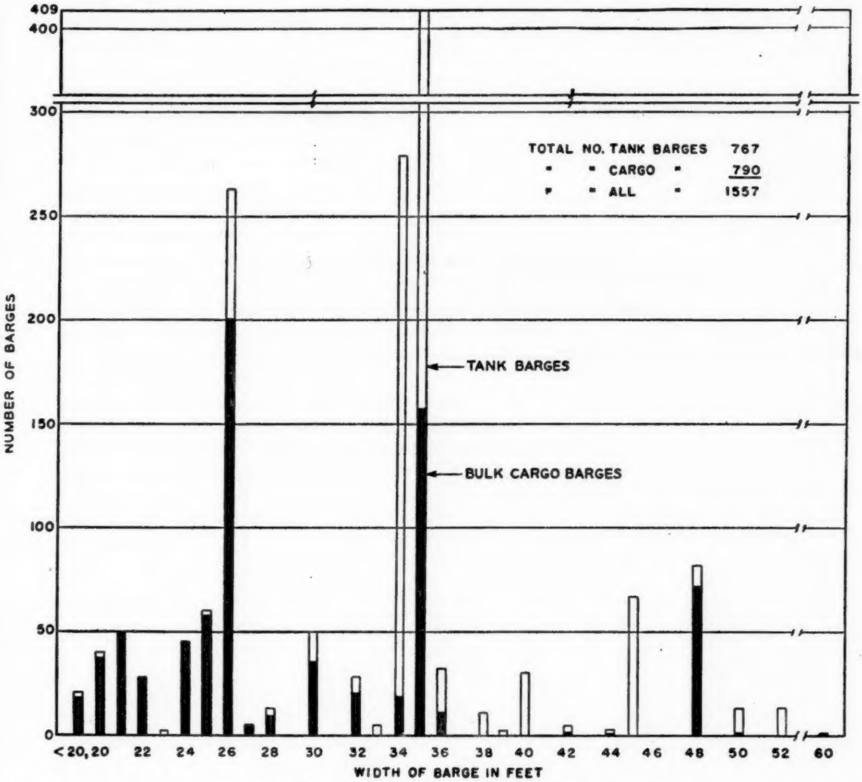


Fig. 19.—Width of Barges on the Mississippi River

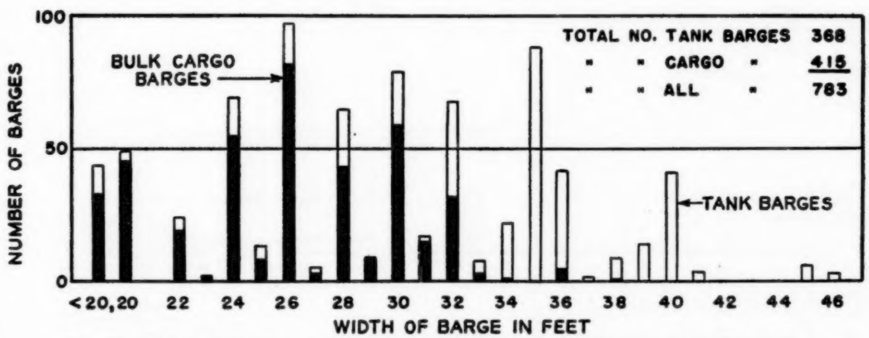


Fig. 20.—Width of Barges, in Feet, on the Louisiana-Texas Intracoastal Waterway

to obtain cargoes from one customer to fill them. They require larger handling facilities to load and unload, and longer landing spaces. It may be noted that these disadvantages are less important for liquid cargoes. Finally, long

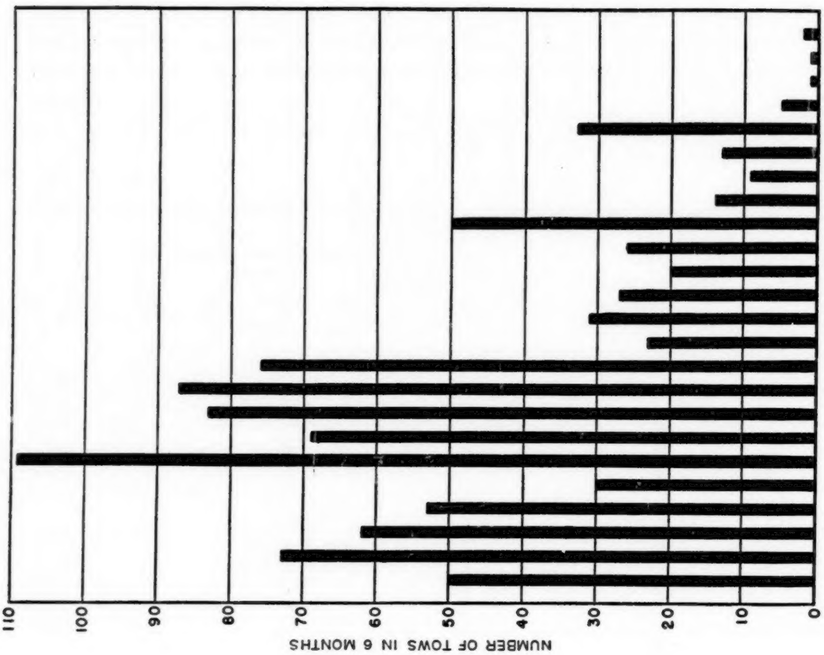


FIG. 22.—BARGES PER TOW, LOCK 36, OHIO RIVER

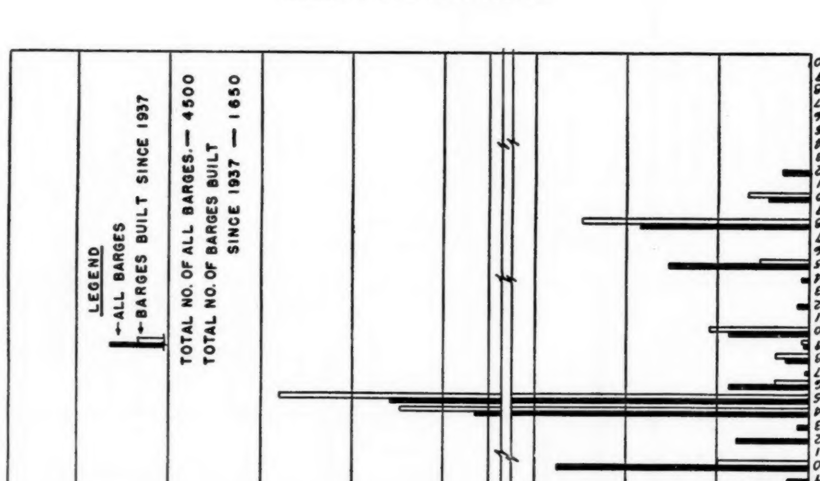


FIG. 21.—TREND IN THE WIDTH OF BARGES, MISSISSIPPI AND OHIO RIVERS

narrow barges that permit the assembly of long, narrow tows have the advantage of requiring less towing power. Model towing tests indicate, for instance, that from 25% to 50% more power is required for equal speeds if a tow of six barges is arranged three wide and two long rather than two wide

TABLE 2.—BARGES OPERATING ON THE MISSISSIPPI AND OHIO RIVERS, IN 1945

Width, in ft	Bulk Cargo Barges				Tank Barges			
	No. in Operation		New Since 1937		No. in Operation		New Since 1937	
	Ohio	Mississippi	Ohio	Mississippi	Ohio	Mississippi	Ohio	Mississippi
<20	24	18		5		3		2
20	26	37	5		4	3		2
21		50						
22	24	28	7	5				
23						2		
24	31	45	6	14	5			
25	3	58		5		2		
26	2,094	200	640	69	41	63	17	28
27	234	5	24		1			
28	34	9	2	3	2	4	2	4
29	9				1			
30	65	35	4	4	9	15	1	7
31								
32	5	20		4	2	8		
33					5			
34	2	18		2	1	261		239
35	136	157	35	102	162	252	128	193
36	3	11		1	4	21		5
37					2			
38						11		6
39						2		1
40					9	30		18
41								
42		1				4		
43								
44		1				2		
45					2	67	2	7
46								
47								
48		72		35	1	10		6
49								
50		1			8	12	3	8
51								
52						13		
53								
54								
55								
56								
57								
58								
59								
59								
60		1						
Totals	2,690	767	723	249	259	785	153	526

and three long. The tests also indicate that short barges assembled into a tow of a certain width and length would have added towing resistance over long barges assembled into a tow of equal dimensions. The additional resist-

ance would come from the drag produced in the larger number of transverse joints in the tow where the raked ends of the barges come together.

The manner in which barges will be assembled into tows is the subject that must be considered in proceeding from barge sizes to lock sizes. It has been mentioned that long narrow tows require less power for equal speeds, and barges will be so assembled if navigating conditions permit. Curvature and width of the navigating channel have a great effect on the practical length of a tow. Although it may be desirable to operate a tow of twelve barges on the Ohio River under very favorable conditions arranged two wide and six long, such a tow could not operate on any Ohio River tributary and probably would not be considered safe for open-river navigation because of higher stream velocities. In restricted channels, having radii of curvature as small as 1,200 ft, it may be necessary to ignore all considerations of towing resistance and arrange tows two barges wide and one barge long for sufficient steerability to negotiate the sharp turns without backing and flanking. Of course, there will be many combinations between these two extremes. Tows are being built up to as many as twenty-four barges on the Ohio and Mississippi rivers. Fig. 22 shows the number of barges making up tows passing through a lock on the middle Ohio River during a 6-month period. The most recurrent size of tow is one that contains six barges, and there is a sharp decrease in the number of tows containing more than ten barges. This trend represents a lock-size effect since most tows of ten barges passed with a single lockage (lock size, 110 ft by 600 ft), whereas a larger number of barges nearly always required a double lockage. There are enough tows containing sixteen or twenty barges to indicate that double lockages are considered worthwhile for this number and possibly to indicate that longer locks or twin locks might be desirable.

Towboats.—The size of towboats has a minor effect on lock size, not only because there is only one of these per tow, but also because improved designing has made it possible to produce towboats of greater power and smaller hull size than in the past. Towboats of fairly recent design, of the class from 2,000 hp to 2,400 hp, now vary in width from 38 ft to 58 ft and in length from 180 ft to 240 ft. It appears likely that boats of this power will be 50 ft by 200 ft or less in the future. Furthermore, slack-water navigation conditions such as are produced by locks and dams are favorable to the use of less powerful towboats than the foregoing, and on waterways of limited width and sharp curvature horsepowers of more than half of those mentioned would be of no value.

Make-up of Tow.—On inland waters of the United States barges are almost always pushed by the towboat. In narrow channels of excessive curvature or in rough water, barges are sometimes towed astern; but it is a dangerous practice because of the uncontrollability of the barges. In the ordinary tow 1 ft for fenders should be allowed between barges in computing the width of the tow and 1.5 ft should be allowed on each side of the tow in computing lock width in order to facilitate entrance. At least 10 ft, and preferably 20 ft, should be allowed in lock length to minimize the possibility of gate damage.

Standardization of Lock Sizes

There has been no particular effort to standardize lock sizes in the United States; but, as waterways become interconnected and as transportation pat-

terns become more fixed, there would be some advantage in doing so. A lock size of 110 ft by 600 ft has established itself as a kind of standard because it is wide enough to accept either the 26-ft barge or the 35-ft barge without wasting space. Since these sizes include the vast majority of barges and since (even on the large navigable waterways) there is an advantage to longer tows rather than to tows of greater width, the size is tolerable even when double lockages are necessary. The lock size of 56 ft by 360 ft has also been widely used although it accepts only 26-ft barges without waste of space. There is nothing that can be foreseen which suggests a change in the 110-ft lock width. If greater tonnages are to be carried in the future, the first step will be to increase depths, and if larger barges become desirable there is no important reason why they should not be approximately 50 ft wide and thus substantially fill a lock 110 ft wide. Transportation experience would seem to indicate that barge sizes will not increase except for certain special cargoes because of the desirability of keeping loads within the limit of individual customer requirements and facilities. Therefore, a lock width of 110 ft is considered an acceptable standard for the future. The length of 600 ft is satisfactory for barge sizes since barge lengths generally are close to, but less than, 200 ft; but for the larger rivers a length of 1,200 ft must be considered since tows of this length have already been operated to a limited extent and future increases in traffic will make them more common. On slightly more restricted waterways a length of 800 ft may have a place. The 56-ft-wide lock is not so promising for the future because it will not accept 35-ft barges properly. To accommodate these and also the 26-ft barges, with the least waste, the next width less than 110 ft should be 83 ft. A still smaller but acceptable width, although it would be accompanied by more wasted space, would be 74 ft, and, since the difference between this and 83 ft is so small, it appears that a minimum lock width of 83 ft would be justified in most cases. Corresponding lengths, depending on the nature of the waterway, would be 400 ft, 600 ft, or 800 ft on the basis that barges as long as 195 ft must be accommodated.

Conclusions

Five general conclusions can be drawn from this paper:

1. Navigation locks are structures capable of long life;

2. Lock sizes adopted in the past frequently have been too small to utilize the full life of the structure;

3. Lock sizes should be determined on the basis of economic studies with consideration given to additional, and sometimes intangible, factors such as (*a*) future increases in traffic, (*b*) undesirability of delays at locks, (*c*) future increases in size of tows, and (*d*) high cost, in many cases, of providing additional or augmented locks;

4. Barge and towboat sizes will not change enough in the future to render the presently acceptable lock sizes obsolete providing the lock sizes are large enough to accept larger tows with less delay;

5. Acceptable standard lock sizes for future construction, in feet, are:

Width	Length
110	600
110	800
110	1,200
83	400
83	600
83	800

References and Acknowledgments

Data have been extracted from publications[10,11] prepared by the Board of Engineers for Rivers and Harbors, Corps of Engineers, and by L. A. Baier.[12]

Advice regarding barge construction was furnished by the Dravo Corporation in Pittsburgh. Statistics regarding tow make-up and movements were supplied by the office of the Division Engineer, Ohio River Division, Corps of Engineers.

[10] "Locks and Dams of the United States, Navigation Data," prepared by the Board of Engrs. for Rivers and Harbors, Corps of Engrs., U. S. Dept. of the Army, Washington, D. C., December 31, 1945.

[11] "Transportation Lines on the Mississippi River System, 1946," *Transportation Series No. 4*, prepared by the Board of Engrs. for Rivers and Harbors, Corps of Engrs., U. S. Dept. of the Army, U. S. Govt. Printing Office, Washington, D. C., 1946.

[12] "The Resistance of Barges and Flotillas," by L. A. Baier, *Transactions*, Soc. of Naval Archts. and Marine Engrs., 1947.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

MOVEMENTS IN THE DESICCATED ALKALINE SOILS OF BURMA

By F. L. D. Wooltorton [1]

Synopsis

Vertical movements in the tropical, desiccated, alkaline soils of the Mandalay District, Burma, are not considerable. An extreme maximum free soil surface movement of 3 in., based on volume changes over a calculated depth of 26.5 ft, is probably less than occurs in Texas. Horizontal shrinkage may be 8% at the surface over soil cracks 3 ft apart. Movements imparted to brick structures, when the dry soil (moisture content approaching or below the shrinkage limit) at foundation depths takes up moisture over the residual shrinkage range (about 5%), are sufficient to lead, by accumulative differential effects, to distortion and cracking when such structures transmit loads less than the effective soil swelling pressure. The deflection curves for such buildings are dome shaped. A maximum building deflection of 2.88 in. was recorded.

The soil was classified agriculturally as a calcium-magnesium-sodium solonchatic-solonetzic complex containing considerable illite and some montmorillonite, and engineeringly as a nonplastic A-6 clay of considerable carrying capacity.

Whereas the relatively high replaceable sodium and magnesium pointed to detrimental volume changes and impermeability, the replaceable calcium and magnesium suggested permeability. Although indicated as impermeable by the permeability test, mechanical, chemical, and moisture-content analysis showed the soil to be permeable. Moisture-content and volume changes thus occur rapidly. Badly cracked buildings were not over soil with higher clay content or shrinkage coefficient but with higher permeability. Within the moisture range, the swelling pressure of the clay is cyclically mobilized producing periodic horizontal and vertical loadings on foundations leading to accumulative movements.

Note.—Written comments are invited for publication; the last discussion should be submitted by June 1, 1950.

[1] Constr. Engr., Road Constr. Div., Public Works Dept., Nakuru, Kenya Colony, East Africa.

Building damage occurs at periods leading up to maximum moisture content (seldom exceeding 19%) and arises from the high reverse deflections whose effects are accentuated by sinusoidal modifications. Buildings transmitting

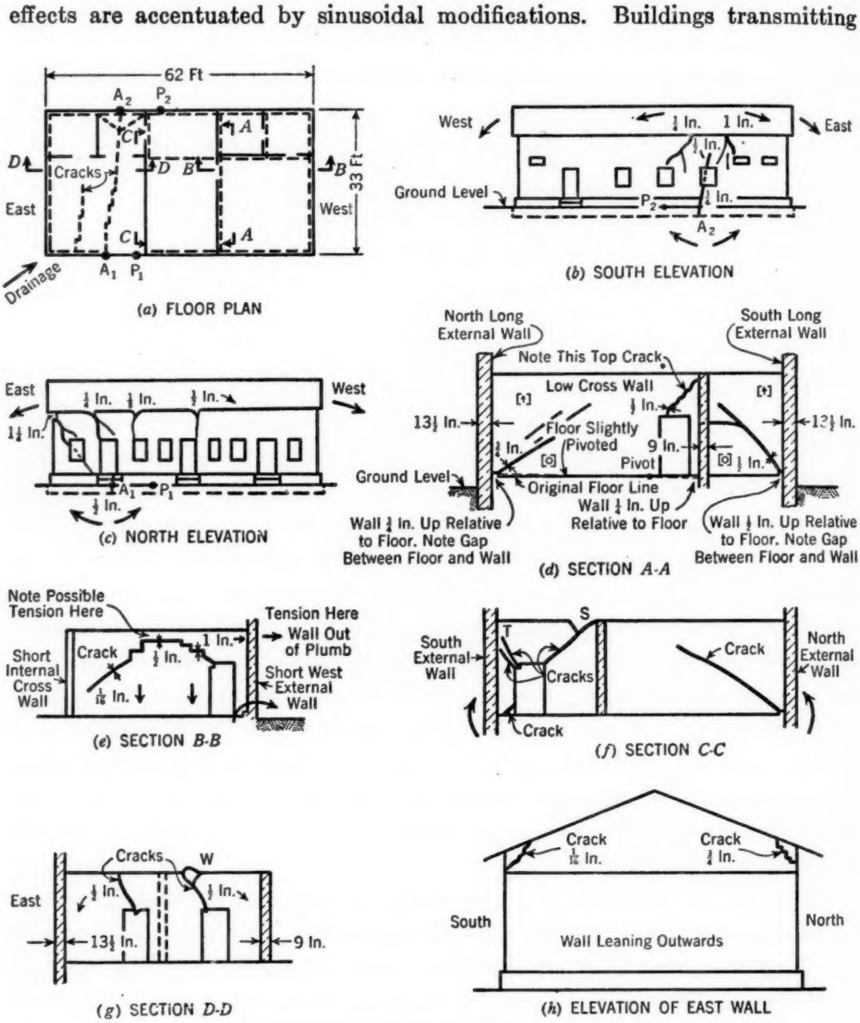

FIG. 1.—OFFICE BUILDING, MANDALAY RACE CLUB, MANDALAY, BURMA

loadings appreciably greater than the swelling pressure were not damaged and their deflection curves were dish shaped.

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1. INTRODUCTION

The study of soil movements in the desiccated soils of the dry zone of Central Burma arose out of a specific investigation into why, soon after con-

struction, buildings in this region developed a most elaborate, and often symmetrical, pattern of cracks. In five known examples the cracking was so persistent and so severe that structures had to be demolished and rebuilt.

The main reason given for the cause of cracking was that, as the soil was considered to be soluble and hence plastic, the foundation loadings were too high. This view had resulted in designs to transmit, in the main, very low load concentrations.

The object of the investigation was principally to discover the phenomena that caused buildings to crack, and, also, why such cracking continued, in many instances, year after year without abatement. An explanation for the phenomena could not be found by pure soil mechanics; the investigation soon led to a pedological study of the soil and later to a study of the building deflection curves.

An example of the basic problem can be demonstrated by Fig. 1, a simple, rectangular, single-story, brick building, in Mandalay. The exterior walls were 13.5 in. thick and 16 ft high, the interior walls were 9 in. thick and 8 ft high, capped by a roof supported on steel trusses. The external walls rested on a cement concrete strip footing which in turn rested on a 12-in. cushion of sand. The underside of the foundation (3.75 ft below the ground level) was in the transition layer, between the surface "black cotton" soil and the lower "kyatti" soil. (Soil types are described, subsequently, in Sections 2 and 3.)

The building had cracked so badly as to be beyond repair. The cracks were as wide as 2 in. and the short end walls on the east were 2.5 in. out of plumb, the tops leaning outward. (In Fig. 1, the east end wall was out of plumb 1 in. at the ends and 2.5 in. at the center. The west end wall was out of plumb $\frac{3}{4}$ in. at the ends and 1.5 in. at the center. The cracks indicated in Fig. 1 illustrate both upward and downward movements of the center. The dimensions refer to the width of cracks, and the arrows show the direction of movement.) Although the walls were deflected, there was no sudden deflection due to uneven settlement and consequent shear. The bearing pressure was only 0.25 ton per sq ft. Levels along the plinth (water table) on the north and the south gave definite curves indicating that the building might be considered to have pivoted about some central axis, P_1P_2 (Figs. 1 and 2), at right angles to the long walls. The maximum variation in levels over the entire foundation was 2.5 in., other observed maxima being:

Wall	Movement (in.)
North—	
Laterally	0.5
Vertically (assuming water table originally level)	2.4
South—	
Laterally	0.75
Vertically	1.68

The variation in level of the long foundation walls toward the west ends was not uniform but indicated a second inflection point. The lowest part was the west end, not the east end which received the brunt of the drainage.

An examination of the cracks revealed three main forms—(1) bending tension, (2) tension by reverse bending, and (3) diagonal tension.

(1) In the long walls there were tension cracks in the tops extending about one third the way down—cracks which were in accordance with the variation in foundation levels and the deduction that the building had pivoted about a central axis, $P_1 P_2$, perpendicular to the long walls, as shown in Figs. 1(*a*) and 1(*b*).

(2) One bending tension crack, near the pivot points in each of the long external north and south walls, was in the bottom of the wall (see points A_2 and A_1, Figs. 1(*b*) and 1(*c*)), suggesting that bending in the reverse direction to that then evident had occurred at some period, although to a lesser degree. This was also indicated by floor cracks, running between points A_1 and A_2 (Fig. 1(*a*)) close to, and approximately parallel with, the line $P_1 P_2$, although the width of these cracks might have been due in part to longitudinal movement. Furthermore, some of the cracks in the top of the long internal low walls suggested compression failure—namely, point W in section D-D (Fig. 1(*g*)).

(3) The points of failure in the 9-in., low, short interior walls were diagonal tension cracks caused by shear. From the point where the cross walls joined the exterior walls a diagonal crack extended three fourths of the way up the cross walls (see Fig. 1(*f*)). Around the entire exterior wall (except for the area near the north wall pivot P_1), and along the cross walls (except in the center of the building), there was a relative raising of the wall to the floor (see Fig. 1(*d*)).

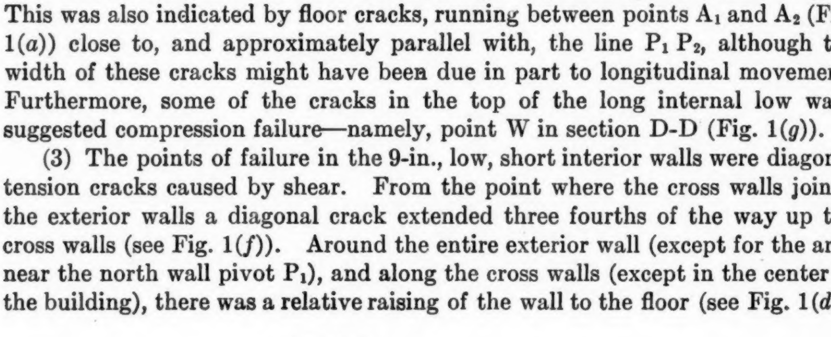


FIG. 2.—SETTLEMENT OF THE FOUNDATION

An examination of about eighty buildings showed that the types of cracks described in the foregoing example were representative of most of the cracks found.

2. DESCRIPTION OF LOCAL SOILS

Two soils were involved known locally as kyatti and black cotton. Neither term is a pedological one, although the latter conveys a little more information to an outsider than the former.

Kyatti Soil.—The term kyatti merely conveys the idea of stickiness pertaining to certain layers in most of the local soil profiles. J. Charlton[2] has remarked that:

> "In actual practice it has been found that soils called 'Kyatti' (sticky) by Burmese cultivators are very liable to give bunds in which holes or caverns appear and, or, which ravine badly under rainfall."

Kyatti is the local name for the sticky soil below, or until comparatively recently below, the surface layer which might have been a local black cotton soil in areas where the relief would permit such soils to be formed. It represents a zone of accumulation of fine material and salts.

In general, kyatti soil has the following properties:

a. It is strongly alkaline, having a pH-value greater than 8.4 and sometimes in excess of 9.75.[3]

b. The presence of soluble salts and replaceable sodium causes the (disturbed) soil to be soluble in running water when the soluble constituents are removed in solution or colloidal suspension.

c. It is highly calcareous (nodules).

d. It lacks organic matter and hence is deficient in protective colloids although rich in dispersible colloids.

e. It is said to be impermeable.

The foregoing properties appear to apply more specifically to well-drained, uncultivated areas of kyatti surface soil.

Black Cotton Soil.—The surface layer (generally from 3 ft deep to 4 ft deep) overlying a kyatti, or accumulation, layer below, is designated black cotton soil. It has a pH-range of from 6.5 to 8.4. The presence of organic matter and the leaching of some of the deleterious salts appear sufficient for kyatti dispersion properties to be absent, but a number of dispersion tests suggested that the locally so-called black cotton soil might have kyatti characteristics in an extreme form.

The origin of the black cotton soils of Mandalay is very different from those of the central provinces and Hyderabad, India. Whereas the former are alluvial, the latter are said to be weathered products of the decomposition of Deccan trap or basalt.[4]

The Indian black cotton soil, or "regur," is claimed to be a highly argillaceous, calcareous, fine-grained soil containing a high percentage of calcium and magnesium carbonates and up to 10% of organic matter. (B. V. Nath[5] throws doubt on this claim and states: "The black cotton soils, or, the 'regurs' do not contain much organic matter.")

Mandalay black cotton soil, as subsequently found, is a siliceous, calcareous, fine-grained soil containing relatively high replaceable sodium and magnesium and only a little organic matter.

[2] "A Note on Soils Regarding Their Suitability for Making Irrigation Works Exposed to Water," by J. Charlton, in "Agriculture and Live Stock in India," Vol. 1, 1931.

[3] "A Note on Soils Regarding Their Suitability for Construction of Irrigation Works, Exposed to the Action of Water," by J. Charlton, *Proceedings*, Assn. of Engrs. in Burma, Vol. 111, 1931.

[4] "Theory and Design of Foundations in Black Cotton Soil," by Y. D. Kumar, *Journal*, Inst. of Engrs., India, Vol. X, 1931.

[5] "Black and Red Soils of India," by B. V. Nath, *Bulletin No. 2*, Indian Soc. of Soil Science, 1939.

Soil Structure.—Throughout the profiles of the soils examined, the microstructure appears single grained. In a strict sense, the soil has no macrostructure; but, nevertheless, cracks form during the dry weather, causing the formation of very large and approximately hexagonal columns. Toward the end of the hot dry weather these cracks are from 2 in. to 4 in. wide, extending to a depth of 7 ft or more. The corners of the hexagons sometimes become detached and sink, leaving an irregular hole which, in time, often becomes a permanent hole or depression.

In disturbed kyatti soil the structure (during the hot dry weather) tends to be loose, friable, and "nutty" at the surface to cloddy at the depths beneath the surface (undisturbed). It is in such soils that roundish vertical piping (about 6 in. to 18 in. in diameter) is found, and in which ravines and caverns form so rapidly.

3. Agricultural Classification

The absence of pedological classification necessitated an attempt to classify the Mandalay soils examined. A monsoon rainfall of 34 in. occurs between May 15 and October 15; the average temperature is 28° C; and there is an apparent mixed tropical podsolic and lateritic weathering system. Under these conditions the soils are believed to be transitional yellow earths occurring north of the tropical laterites of Lower Burma and south of the believed brown forest soils of Upper Burma. In particular, they appear as "intrazonal-secondary-solonchatic-solonetzic" complexes although drainage is generally free and no obvious permanent water table exists for a considerable depth. It appears, however, that a temporary water table exists within a depth of 30 ft during the rains, as evidenced by shallow wells (30 ft deep), and within a depth of 12 ft toward the end of the hot dry weather, as evidenced by moisture-content data, despite the fact that air is always present.

The profile is reasonably consistent in the areas studied, and consists of a 3.5-ft-deep, dense hard layer of black cotton soil, overlying a dense hard accumulation zone (4 ft deep) of black to yellow, or mottled black-yellow kyatti soil. A number of "sticky" horizons may be found in any deep profile. At deeper strata lies the dense yellow parent alluvium extending below a depth of 100 ft where it overlies sand.

The soil is mildly siliceous (silica-sesquioxide ratio about 2.6) indicating cohesion but little plasticity. The predominant replaceable base varies and may be calcium, magnesium, or sodium. The stickiness of kyatti soil is due to the presence of sodium salts and replaceable sodium in the clay complex. The relatively high molecular ratio of replaceable sodium to replaceable calcium justifies the clay being generally classified as one that has the properties of a sodium clay. For such a weathering system, the high replaceable magnesium probably also plays an important role in the properties of the soil. It indicates both expansive properties and aggregation, or perhaps permeability, and the aggregation would be accentuated by the presence of the free and replaceable calcium.

The clay minerals for samples taken from a depth of 21 in., and examined by X-ray methods by George L. Clark, were a small amount of montmorillonite and, apparently, considerable illite.

Such a clay would be expected to exhibit considerable and rapidly attained swelling and shrinkage, high density, high cohesion, and low plasticity. Hence, it would be a clay of considerable carrying capacity. It would be expected to differ from a pure sodium clay, however, in its diffusive properties and moisture-holding capacity, in place, because of the flocculating effect of calcium and magnesium. This flocculation might be sufficient to prevent impermeability which is associated with sodium clays.

The untreated and pretreated mechanical analyses examined, in conjunction with the chemical analyses, had also indicated aggregation. They showed that the dispersed clay content increased from about 25% in the surface (loam) layer to a maximum of 61% at the 3.5-ft (clay) layer, and thereafter decreased to 48% at the 12-ft (clay) depth, with an accumulation layer between 3 ft and 7 ft marking the limits of upward and downward weathering. Clay-loam lenses were occasionally recorded between the depths of 8 ft and 12 ft but their thicknesses appear to be limited to about 1 ft and, from the many borings taken, their plan dimensions are believed to be small.

To the depth of 12 ft examined, and especially that in the accumulation band, the soil might have been assumed to be impermeable (as generally believed); but from the analyses it was found to be aggregated, with a maximum aggregation at the depth of the maximum clay content—that is, 3.5 ft. This aggregation was not visibly obvious. By these analyses, the depth of 7 ft (not that of 3.5 ft) was indicated to be the least permeable. This finding was supported by the observation that the maximum apparent density of 2.4 and the maximum pH of 9.1 occurred at a depth of 7 ft.

4. Soil Moisture

As it was considered that structural damage was correlated with moisture changes, arrangements were made to determine the monthly variation in moisture content for the depths, 3.5 ft to 4.5 ft, 6 ft to 7 ft, and 11 ft to 12 ft, at four residential areas to determine the maximum and minimum values and to observe whether a zone of constant moisture content existed within these depths (Fig. 3). In addition, the percentage,

$$S = \frac{\text{moisture}}{\text{saturation}} \times 100 \quad \text{(1)}$$

was determined. In this percentage value, the denominator was determined for a dried pulverized sample. Mean values of S are plotted in Fig. 4 for three representative residential areas. The average rainfall was 34.36 in.; the mean annual temperature, 82° F; and the total rainfall for the twelve months tested, 34.87 in. Within this period a load test (see Fig. 4) of 2 tons per sq ft was applied to kyatti sodium clay at a depth of 5.38 ft. The intention of the second measurement for determining S was to allow for any variation in soil texture which might occur in the monthly samples extracted from any particular depth.

The second task was to estimate the effect of seepage water, chemically comparable with rain, when the soil was in a condition similar to the thin layer immediately below local shallow foundations—that is, slightly puddled.

5. Summary of Soil Moisture Observations

The results of soil moisture observations can be itemized in three groups under "direct results," "indirect results," and "the effect of water on disturbed Mandalay soils."

Direct Results for the Residential Area.—

1. Monthly readings to a depth of 12 ft showed there was no zone of constant moisture content throughout the year and, hence, that some potential movement must be experienced by any structure whose foundations are laid

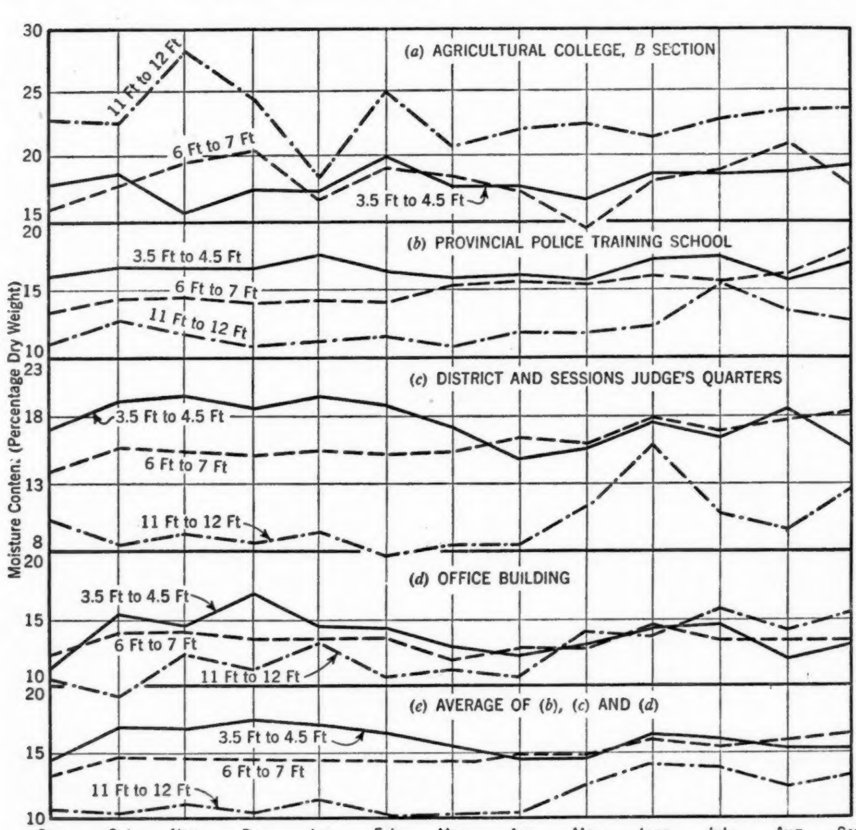


Fig. 3.—Yearly Variation in Moisture Content, September, 1935, to September, 1936

within this depth. Nevertheless, the observations showed the undisturbed soil to be saturated for much of the year although this condition (as will be shown) is accompanied by an entrapment of air.

2. A "good" site, as judged by absence of cracking, is indicated by minimum moisture variations and minimum rates of change of moisture. In other words the value of the site decreases as its permeability increases.

3. The variation in, and the rate of change of, moisture content tended to be at a minimum at the depth of 7 ft (maximum pH-values), showing this level to be, in general, the least permeable and, by virtue of lower and slower moisture changes, the most suitable for foundations.

4. During the period from March to April, before the rains broke, the moisture content began to increase between 12 ft and 6 ft, attaining maximum values in May, which was still before the rains broke. This anomaly (see, subsequently, in Section 18) can be explained by the "distillation effect"[6,7] which, for arid soils, led to the first horizon of ground water, in summer (dry weather). In other words, during dry weather, a temporary perched water table was formed—by condensation, and not by rain.

5. The rapid changes in moisture content during the rains suggest that the soil may be permeable to rain water.

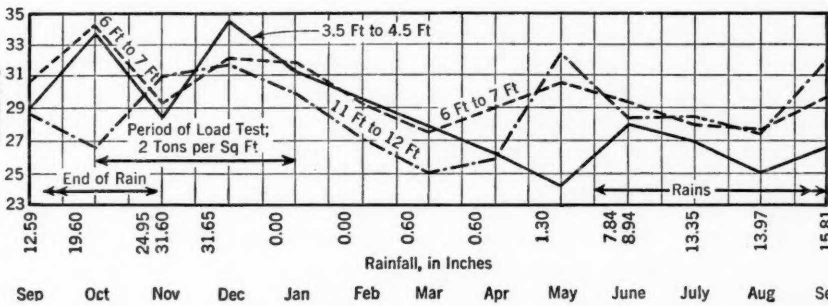


FIG. 4.—VARIATION IN MOISTURE SATURATION CAPACITY, SEPTEMBER, 1935, TO SEPTEMBER, 1936

Indirect Results.—

6. The indirect results of the moisture data were not immediately apparent, principally because the local belief that the soil was a sodium clay and, therefore, impermeable (supported by the high dispersion factor and high apparent density) had not been doubted. When the permeability of the undisturbed soil was at last suspected, the moisture values became of great interest; the key was found in the light rainfall for July and August preceding the first moisture determination in September and in the question of what happened to the heavy rainfall (7 in.) occurring in September. If the undisturbed soil were permeable, this percolating water would flow through the profile under the influence of certain laws and restrictions, but if it were impermeable the moisture would be retained in one of the layers. The heavy rainfall in September continued during October (5.21 in.) and November (7.87 in.), affecting the results for November and December.

The Irrigation Department of Burma could not give the runoff for local uncultivated areas but stated that it could not exceed 30%. Leonard C.

[6] "The Movement of Ground Water and Soil Water," by A. F. Lebedev, *Proceedings and Papers*, 1st International Cong. of Soil Science, Vol. I, 1927.

[7] "Relation of Water to Soil," by A. F. Lebedev, 2d International Cong. of Soil Science, Vol. VI, Comm. VI, Moscow, 1930.

Urquhart,[8] M. ASCE, quotes the late E. Kuichling, M. ASCE, as giving a relative imperviousness of 0.05 to 0.25 for lawns and meadows,[9] the value used depending on the surface slope and character of the subsoil. Therefore, a value of 20% would be accurate enough for this investigation. For evaporation losses, an interpolation was made from the values found by A. D. Hall[10] as quoted by G. W. Robinson.[11] This suggested a maximum loss of 2.7 in., or 39%, at the prevailing average temperature during the rains. This value is a maximum because Mr. Hall's data were determined for a tilled surface, whereas in Mandalay the surface is covered by a slight mulch which, according to some authorities, tends to prevent evaporation. As a check, the formula of Walther Leather (as given by Harry R. Kempe[12]), involving temperature, humidity, and wind velocity, gave an evaporation of about 1.5 in., or 21% of the rainfall. The plant transpiration factor for uncultivated land is normally small and negligible in Mandalay where, for the areas examined, plants virtually do not exist.

Thus, for the month of September, 1935, out of the 7 in. of rainfall only about 4 in. (approximately 57%) can be accounted for by losses and the remaining 3 in. must have gone into the soil. The comparative constancy of the moisture contents, as regards the high values which were below the field moisture equivalent (FME), indicated the soil to be permeable.

Consideration of the problem from the numerical values revealed that, although the moisture content at any depth was not constant, for certain months there existed a surprising degree of constancy except for the 11-ft to 12-ft level where the soil was less homogeneous. Studied with the rainfall, it will be seen that during July and August there was little rain; but the heavier rainfall of 7 in. in September was reflected in an increase of moisture up to approximately the "field capacity" for the October values—in two instances down to 7 ft (the limit of downward weathering), and in the third down to 12 ft. The average increase for the three areas, allowing for the decrease at the 12-ft level in two areas, was 0.95%, 0.41%, and 3.29%, or an over-all average increase of 1.55%. This increase corresponds to $1.55/100 \times 12 \times 1 \times 100 = 18.6$ lb, or 3.5 in. of rain, against the value of 3 in. computed previously as having entered the soil. This comparison suggests that, on the average, all the available rain had percolated the profile and was just sufficient to reach a depth of 12 ft in one month. These values give an average movement of rain water, in a downward direction, of 5 in. per day.

For October, November, and December, when the rainfall was 5.21 in., 7.87 in., and 0 in., respectively, the moisture contents, after their initial increase in October (which, in two cases, was delayed for the 12-ft depth), remained constant (with one exception) at 3.5 ft to 4.5 ft and 6 ft to 7 ft and nearly constant at 11 ft to 12 ft. Such constant values show that, for such a

[8] "Civil Engineering Handbook," by Leonard C. Urquhart, McGraw-Hill Book Co., Inc., New York, N. Y., 2d Ed., 1940, p. 793.

[9] *Transactions*, ASCE, Vol. LXV, December, 1909, p. 400.

[10] "The Book of the Rothamsted Experiments," by A. D. Hall, Rothamsted Experimental Station, Harpenden, Hertsford, England, 1905.

[11] "Soils, Their Origin, Constitution and Classification," by G. W. Robinson, Thomas Murby & Co., London, 1936.

[12] "Engineer's Year-book," by Harry R. Kempe, Morgan, London, 1939.

soil, the moisture content during these months represented the Veihmeyer or "agricultural field" capacity (the volume of water held in the soil after excess gravitational water has drained away) and that, to a depth of 12 ft, the soil was saturated under field conditions. Otherwise the moisture would have continued to increase. During January there was no rain and the results for February and March indicated a general decrease.

The writer considers that the foregoing observations prove that the undisturbed soil is permeable and that, for much of the year, it is in an undisturbed state of saturation. This was not visibly obvious because of the high density of the soil and the percentage of entrapped air. The continuance of fairly constant saturation can only be explained by the fact that the field capacity is nearly equal to the limit of "osmotic imbibition" (defined as the maximum moisture content that clay particles will adsorb under osmotic type of forces which, for a highly expansive clay, approximates the FME). This argument is supported by the rapid and considerable changes in S that occur during the rains, as shown by Fig. 4. In other words, except when water is actually percolating the profile, there is generally little capillary water. This argument was corroborated by consolidation data.

7. Furthermore, it follows from the definition of "field capacity," when applied to a permeable soil, that it must be approximately equal to the "shrinkage limit" of undisturbed soil.

The Effect of Water on Disturbed Mandalay Soils.—

8. A site adaption of the slaking test, with remolded samples, showed that in this condition the deeper the soil (to 12 ft) the more slowly it dried and absorbed water. This finding was supported by the wetting times of the previous experiment on pulverized soil for the determination of the saturation capacity. The inference is that, when disturbed, the soil's resistance to changes in moisture content by fairly pure water increases with depth to 12 ft and that its permeability decreases with depth. This is in fair agreement with the discussion on the chemical analysis (see Section 3) but, as in that discussion, it leads to results contrary to what might have been expected from a study of the mechanical analyses alone.

9. It may appear also that these results are inconsistent with items 6 and 7, but it must be remembered that conditions are not the same in the two experiments. In items 6 and 7 variations in moisture are due to soil, or saline water, and are caused by natural changes in the moisture content of a soil with an undisturbed structure. In this test on disturbed soils the results are due to relatively pure water, such as might occur during rain when water percolates between the building and the soil. In the former instance the flocculated condition is preserved; in the present instance the water apparently deflocculates the remolded and structureless clay, below 7 ft, causing impermeability.

10. If the soil at foundation depths is superficially puddled, it will thus resist the seepage water entering between the soil and the foundation wall; and, for the short periods of local rainfall, the water is likely to have little effect. Once through the puddled and relatively impermeable layer, the water

will be led away by the increase in permeability. Thus, little building damage may be expected as a direct result of percolation.

6. Relation Between Climate and the Cracking of Buildings

From observations and records of the cracking of buildings it seemed that some cracks began to appear during August. In October and November they became obvious, and had been known to widen in November and December, but the worst month for cracks was undoubtedly May. Records further showed that cracking had been more pronounced during certain years. Therefore, it was decided to examine these records with meteorological records and the moisture content data.

The results clearly showed that there were two main periods during which the cracking appeared—at the end of the rains, and at the end of the dry weather. The former period, occurring during or after a rapid series of maximum and minimum moisture contents, corresponded to a period of maximum moisture content, maximum humidity, and minimum mean temperatures. The latter period occurred when the dry weather minimum moisture content was reached at higher levels, accompanied by a rapid moisture increase between 12 ft and 7 ft. At the same time the mean temperatures were a maximum and the humidity was a minimum.

Cracking, therefore, appears to be caused by volume changes and to occur at periods of minimum and maximum soil volumes. It is augmented by the rapid changes following these conditions. Data showed that the cracking of buildings was most noticeable at the end of a long dry period following a long wet period. The periods investigated were each of two-year duration. Cracking was also found to be associated with periods of exceptionally heavy periodic rain. The moisture conditions of the foundation soil at the time of construction probably have a considerable influence over the future history of the structure.

7. Loadings Transmitted by Existing Structures and Site Loading Test

When it was found that the foundations of the condemned Mandalay Race Club office (see Fig. 1) were only transmitting 0.25 ton per sq ft, it was decided to investigate other loadings to correlate building cracks with loading and depth of foundations and to ascertain if there were any truth in the belief that the soil was being overloaded. It was also arranged to make a site loading test to determine the maximum load the soil could safely withstand, to estimate the accompanying sinkage, and to verify the belief that the plastic yield of the undisturbed soil was small.

An analysis of the loadings transmitted by sixty-six buildings showed:

a. A variation in loading transmitted to the soil of between 0.25 ton per sq ft and 2.1 tons per sq ft;

b. No direct correlation between cracking and depth of foundations; and

c. A direct connection between loading and cracking; few cracks were found in buildings transmitting more than 1 ton per sq ft; and for a loading less than 1 ton per sq ft, and foundation depths between 3 ft and 7 ft, all

the buildings had cracked and thirty two out of fifty five were described as badly cracked. This refuted the belief that cracks were due to overloading the soil.

A confined site loading test was made on an area of 1 sq ft, in a hole about 15 in. square, at a depth of 5.38 ft and with a soil moisture content of about 15%. It was thus determined that (see Table 1), under its maximum normal moisture content, the soil could safely carry 2 tons per sq ft with little settlement (0.36 in.) and no plastic yield. For conditions corresponding to percolation between building and soil, and for flooding sufficient to create a 5-ft head, there was some immediate increase in the settlement (0.09 in. and 0.16 in., respectively) but no continuous and accumulative plastic yield. This proved the soil to be definitely nonplastic and refuted the belief that excessive sinkage was caused by rain and percolation.

TABLE 1.—SETTLEMENT OBSERVATIONS ON UNDISTURBED KYATTI SOIL, UNDER A LOAD OF 2 TONS PER SQ FT

Period	Condition of pit	Settlement[a] (in.)	Weather	Period	Condition of pit	Settlement[a] (in.)	Weather
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
October, 1935				November, 1935:			
8 to 9	Dry	0.36[b]	Fine	3	....	0.44	Rain
9 to 12		0.36	Fine	4	....[c]	0.45	Rain
12 to 26		0.36	7-in. rain	5	Flooded to depth of 5 ft	0.53	Rain
26 to 31		0.36	Fine	6		0.61	Rain
31	0.5 in. of water added daily	0.36	Fine	6 to 12		0.61	Fine
November, 1935:				12	No[d]	0.61	Fine
1		0.44	Fine	12 to 20	Muddy	0.61[e]	Fine
2		0.44	Fine	20 to			
3 to 7	....	....	5-in. rain	February 4, 1936	Dry	0[f,g]	Fine

[a] Cumulative, total settlements. [b] As load increased from 0 to 2 tons per sq ft the settlement increased from 0 to 0.36 in. [c] 1 in. of water added. [d] Water pumped out. [e] Load 1 ton per sq ft. [f] No settlement, but some evidence of cyclic movements. [g] Load 0.75 ton per sq ft.

The possibility of cyclic changes in the levels of the settlement indicator pegs and bench mark on a near-by post footing was suggested and the phenomenon was examined in detail at a later date (see Sections 18 and 19).

8. SOIL CONSTANTS AND CLASSIFICATION

Mechanical Analyses.—A compilation of all data to show the variation in the clay fraction and the soil constants with depth is given in Tables 2 and 3, respectively.

Apart from the shrinkage data, the most interesting values are those for the plastic limit (PL) and the FME, when compared with the moisture contents of undisturbed soils. As the FME (which is held to represent the maximum quantity of water that the disturbed soil can freely hold and to be a value greater than the undisturbed soil may hold) is less or equal to the PL (which is greater than the maximum moisture content of undisturbed soil), the soil will remain stable during wet weather, there will be no plastic yield, and the bearing

value will be unimpaired. This observation was confirmed by the load test reported previously in Section 7.

General considerations of the data and comparison with the old correlation curves issued by the United States Public Roads Administration place the soil in the A-6 group.[13] From correlation curves of the coefficient of friction and the PL reported in 1938,[14] the angle of friction would be 16°. It is probable that, with its low moisture content, the soil retains its granular nature in shear and that this angle is always effective for undisturbed soils.

TABLE 2.—PROFILE CLAY FRACTIONS, WITH AND WITHOUT HYDROGEN PEROXIDE PRETREATMENT

DEPTH (FT)		CLAY (%)		COLLOIDAL CLAY (%)		Soil type
From:	To:	Not treated	Pretreated	Not treated	Pretreated	
0	0.5	20.0[a]	....	10.0[a]	....	Black cotton soil
2	3	39.5	58.4[b]	16.5	49.0[b]	Black cotton soil
3	4	....	61.0	....	....	Kyatti soil
3.5	4.5	47.2[c]	....	44.7[c]	....	Kyatti soil
6	7	44.4[a]	....	39.0[c]	....	Kyatti soil
7	8	....	54.2[b]	....	44.2	Parent soil
11	12	38.7[b]	....	34.0[b]	....	Parent soil
12	13	....	48.4	....	....	Parent soil

[a] Mean of fifty-seven samples. [b] Mean of two samples. [c] Mean of three samples.

TABLE 3.—SOIL CONSTANTS

Structure (as identified in Fig. 3)	Lower liquid limit	Lower plastic limit	Plasticity index	SHRINKAGE:			MOISTURE EQUIVALENT		Specific gravity[b]	Volumetric change	Linear shrinkage[c]
				Limit (remolded)	Ratio[a]	Residual	Centrifuge	Field			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Office Building:											
2 ft to 3 ft deep......	67.4	19.0	48.4	9.1	2.10	3.5	42.3	18.6	2.68	20.0	6.0
7 ft to 8 ft deep......	70.0	20.1	49.9	11.0	2.06	3.5	45.4	20.2	....	18.9	5.8
Quarters of District Judge:											
2 ft to 3 ft deep......	81.1	24.3	56.8	....	....	....	51.4	20.5	....	....	....
12 ft to 13 ft deep....	56.2	18.3	37.9	....	....	....	....	....	2.70	....	....

[a] Approximate apparent density. [b] Specific gravity of soil particles. [c] On wet length.

An examination of considerable data pertaining to many soils has indicated that trouble may be expected, in both building and road construction, when the FME materially exceeds the PL—that is, when there is considerable osmotic swelling. Many such soils had constants identical with those for Mandalay except in respect to the FME and the apparent density. The former were actively plastic; the latter were not plastic despite their high plasticity indexes.

[13] "Classification of Soils and Control Procedures Used in Construction of Embankments," U. S. Public Roads Administration, Vol. 22, No. 12, February, 1942.

[14] *Paper No. 38*, 8th International Roads Cong., The Hague, 1938.

Possibly the difference in the plasticity and in the FME is reflected in the fact that the FME also is a measure of the activity of the soil colloid after drying, when low values indicate a colloid undamaged by drying and high values indicate an irreversible colloid; or perhaps low values are a reflection of the structure and the air-entraining nature of the clay. The FME would certainly appear to represent a measure of the total energy available for absorbing moisture under the colloidal and structural properties of the soil at the time of testing.

9. The Shrinkage Curves

The phenomena of swelling and shrinkage are of more importance to the soil engineer than is realized. The reconstruction of the site shrinkage curves (that is, curves for undisturbed material) is demonstrated by reference to Fig. 5. Consider the depth 3.38 ft at the office compound first mentioned in Fig. 3. The apparent density ρ for an undisturbed sample for the depth was

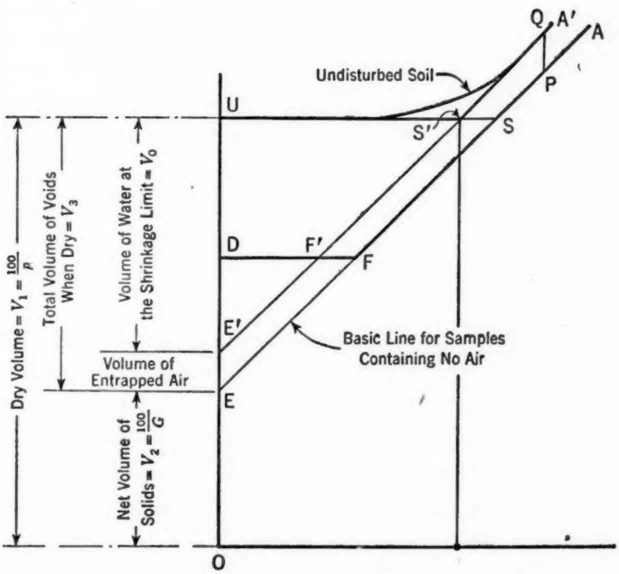


Fig. 5.—Construction of Typical Shrinkage Curve

1.61 and the specific gravity G for the soil particles was 2.68. The field capacity is 14.71% dry weight, in a specimen of 100 g. Therefore, the volume of water at this field capacity, in cubic centimeters, is $V_0 = 14.71$; $V_1 = 62.20$; $V_2 = 37.30$; and $V_3 = 24.90$. In the foregoing, V_3 is the undisturbed site shrinkage limit for no entrapped air. As a check on V_3, when the density ρ equals 1.61 and the void ratio e equals 0.665, the shrinkage limit is the volume of the voids, or $0.665 \times 37.3 = 24.8$ cu cm. The shrinkage limit for undisturbed soil (line SU, Fig. 5) is approximately the field capacity for a permeable soil.

The volume of entrapped air at a moisture content of 14.71% (representing the site shrinkage limit when air is present) is $(V_3 - V_0) = 10.19$ cu cm. If the soil be such that some capillary water still exists at the estimated field capacity, the desiccated shrinkage limit for the undisturbed soil may be slightly less than 14.71. From the foregoing data the reconstructed site shrinkage curves may be drawn, as shown in Fig. 6, in which point E indicates the remolded shrinkage limit, point E_1 is the corrected shrinkage limit, point E_2 is the undisturbed shrinkage limit for the condition in which there is no entrapped air, and point E_3 denotes the actual undisturbed shrinkage limit.

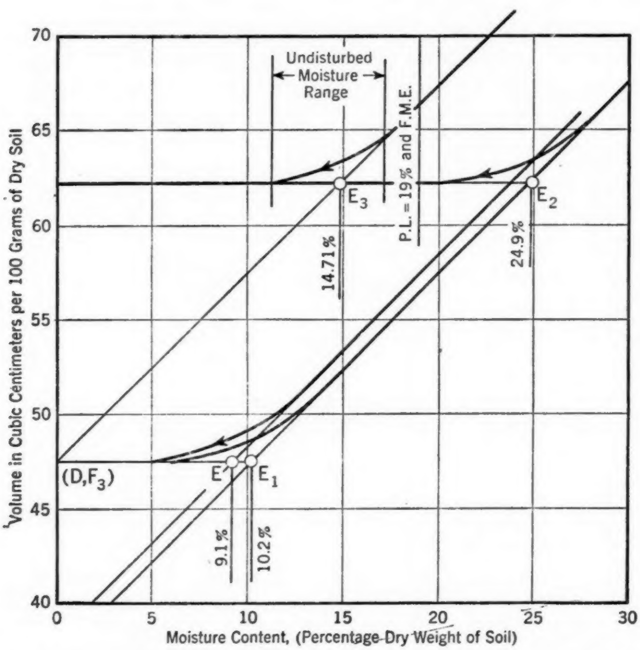


FIG. 6.—RECONSTRUCTED UNDISTURBED SHRINKAGE CURVES FOR OFFICE COMPOUND

The total volume of voids, as represented by the fully saturated shrinkage limit (w_s) of undisturbed material, is 24.9 cu cm. For instance, this is the volume when the material is dried from the fully saturated condition, which can only happen when water is forced in to replace entrapped air. Similarly, the total volume of voids in the desiccated soil, when dry, is likewise 24.9 cu cm of which 14.71 cu cm represents the voids set free by the water held at this shrinkage limit, and 10.19 cu cm represents "entrapped air" or structure.

A point on the site desiccated curve cannot move to its similar position on the fully saturated curve unless the entrapped air is released. If it were not for its structure the soil would be plastic and this condition represents a possible important difference between some soils in the A-6 and A-7 classifications. Diagrammatically the structure (which has gradually been built up and possibly

made more permanent by a form of cementation), could be represented by many cycles of drying (below the shrinkage limit) and wetting between the time of the first fully saturated shrinkage curve and the present curve. This structure is believed to be a very important property of the soils in the classification A-6 examined:

Percentage of entrapped air = 16.2, dry volume
Percentage of pore space = 40.0, dry volume

The undisturbed moisture content range lies within the residual shrinkage range in Fig. 6.

Shrinkage from Undisturbed and Disturbed Curves.—As the site moisture content apparently varies over the residual shrinkage range (see Fig. 6) any changes in moisture content will result in volume changes as determined by these ranges. If the fluctuation of 8.12% (which is found at the 11-ft to 12-ft depth at the quarters of the district and sessions judge) occurred above the residual shrinkage range, it would account for a variation of 8.12 cu cm per 58.5 cu cm (approximately) of dry soil, or a volume change of 13.8%—that is, it would provide for an approximate linear variation of 4.5% on the wet length, whether the soil were remolded or undisturbed. However, as the variation occurs in the residual shrinkage range, the formula for volumetric change gives values that are too high, and the corrected values would be about 11.1% (volumetric) and 3.7% (linear), respectively.

A comparison of percentage changes, volumetric and linear, for undisturbed soil, as determined from various curves, is as follows:

Curve	Volumetric	Linear
Shrinkage	7.0	2.3
Compression (wetting from 1.6 tons per sq ft to 0.0 ton per sq ft)	7.3	2.4
Swelling pressure	9.0	3.0

The agriculturally determined linear shrinkage for the same depth was 3.0% for a remolded sample drying from the sticky point.

It should be noted from the consolidation (rewetting) curve that, although the volume changes are relatively small, the corresponding internal pressure changes over this moisture range are considerable.

10. Summary of Engineering Studies

Undisturbed soil, classified as group A-6 flocculated cohesive clay of high carrying capacity—nonplastic, expansive, dense, easily consolidated, and capable of absorbing water only when manipulated—is unimpaired by natural increases in moisture content. At foundation depths the soil has a voids ratio corresponding to that required to carry a load of 6 tons per sq ft when fully saturated, although the presence of air, free to escape, may mean a slightly smaller value. Comparative shrinkage studies gave the results listed in Table 4.

As moisture changes occur within the residual shrinkage range, high capillary suction may be expected in the field when the soil is in contact with a free water

face. This is supported by the low shrinkage limits, high densities, and the consolidation swelling curve.

High density values show a high degree of consolidation. The effect of structure is to reduce the density by some 25% and to increase the saturated disturbed shrinkage limit by an average of about 100%.

TABLE 4.—Shrinkage Characteristics at Various Depths

Description	3.5 ft to 4.5 ft	11 ft to 12 ft
Percentages:		
Entrapped air (on dry volume)	16.2	21.5
Undisturbed shrinkage limit (air present)	14.7	8.95
Undisturbed shrinkage limit (air absent)	24.9	21.5
Disturbed shrinkage limit (air absent)	10.2	12.5[a]
Apparent Densities:		
Undisturbed soils	1.61	1.71
Disturbed soils	2.10	2.02[a]

[a] Approximate values.

Undisturbed linear shrinkages vary from about 1½% at depths of from 3.5 ft to 4.5 ft to about 3½% at depths of from 11 ft to 12 ft. These values, limited by moisture changes, are not high; but, occurring in the residual shrinkage range for the wetting curve, they are accompanied by high internal pressure changes. Naturally, agricultural shrinkage values were somewhat higher and gave lower values for a "bad" site than for a "good" site.

Experiments to determine the ratio between horizontal and vertical shrinkage were inconclusive but nevertheless indicated values greater than unity.[15]

From the consolidation curve, for depths from 12 ft to 13 ft, it was deduced that the maximum possible undisturbed natural moisture content was 26.1% (or 13.6% allowing for entrapped air) against the recorded maximum of 16.05%, thus confirming the limited capillary water and the presence of gravitional water at the end of the dry weather.

The permeability (by test) of 3.5×10^{-8} cm per min at a load of 2 tons per sq ft appears low for a liquid limit of 56.2, but it is considered that this test does not give true values for undisturbed soil. An analysis of rain-water movements, based on the moisture curves and the observed change in direction of swelling as evidenced by building cracks, gave the following velocities through undisturbed soils (in feet per month):

Direction	Velocity
Downward	12
Upward	5
Horizontal	2

These values show that the moisture movements caused by the early heavy rains cannot reach the central core under a structure until about the beginning of the hot weather.

[15] "Vertical and Horizontal Shrinkage of Black Cotton Soils at Mandalay, Burma," by F. L. D. Woolterton and A. T. Sen, *Current Science*, Vol. XI, 1942.

11. Chemicocolloidal Swelling Pressure

From the moisture variations of Fig. 3 it is clear that the voids ratio must change during the year, even to depths of 12 ft. Any change in voids ratio implies (as in the consolidation curve) a change in externally applied pressure, or (as in the shrinkage curve) a change in the internal forces. The externally applied forces on undisturbed material remain constant, but the internal forces vary with change in moisture content—their effect being equal to a change in external forces. Swelling is equivalent to a reduction, and shrinkage to an increase, in external compression.[16] During swelling any applied loading tends to suppress the swelling pressure; during contraction the applied load aids the volume decrease. This research deals with the equivalent reduction in applied pressure arising out of swelling, or the pressure necessary to stop swelling when the soil is in contact with water. Unless it is distributed evenly under a building, swelling has the effect of an applied load on the structure.

If the change in voids ratio is uniform over a site, any movement or deflection will be uniform; but, if the site is partly covered by a building, any change in voids, resulting in the moisture changes, cannot be uniform over the covered area because of the time lag in moisture movement. It is clear, therefore, that, not only will any movement in the structure be of a differential nature, but the swelling can only be prevented if the soil is loaded to a value greater than the swelling pressure. With more usual loads of, say, from 2 tons per sq ft to 3 tons per sq ft (which are greater than the swelling pressure of this soil, or for a soil that is always fully saturated), the swelling pressure effect is entirely suppressed and movements due to shrinkage (which for the capillary range of Fig. 5, correspond to low pressure changes) are "ironed out" by capillary flow under external pressure, and thus occur more uniformly. It is only for the residual shrinkage range that moisture changes under low external loading become important.

An estimation of the maximum value of the swelling pressure has been found possible by four independent methods:

Swelling Pressure Determined from the Reconstructed Undisturbed Shrinkage Curve Data.—When the moisture content is equal to the engineering shrinkage limit, marking the minimum value of the voids ratio for samples whose line US′ (Fig. 5) is horizontal, the voids ratio is not less than $\frac{21.5}{27.0} = 0.581$. On the consolidation curve for fully saturated soil this value corresponds to a load not greater than 2.2 tons per sq ft—that is, when (wetting under no external load from the shrinkage limit) an applied load not greater than 2.2 tons per sq ft is necessary to prevent swelling in a free surface state. At the test depth of 12 ft the approximate loading required would be about 1.7 tons per sq ft. If an allowance is made for the entrapped air, in the undisturbed soil, the pressure required may be somewhat less.

Swelling Pressure Determined from a Failure Record.—The failure of a cement concrete tie beam, 24 in. by 15 in. by 8 in., connecting two pile tops,

[16] "Principles of Soil Mechanics: Part I. Phenomena of Cohesion of Clay," by Charles Terzaghi, *Engineering News-Record*, November 5, 1925, p. 742.

had been recorded during the construction of a building at the Agricultural College in Mandalay. This occurred after a night of rain and was obviously caused by the expansion of the soil under the beam which had only been reinforced in the bottom. Computations show that the swelling force necessary to crack the beam must have been greater than 1.07 tons per sq ft.

Swelling Pressure Determined by Special Test.—The Building Research Station in Watford, England, devised a test to measure, directly, the swelling pressure of a saturated undisturbed sample. Results indicate a value of between 1.5 tons per sq ft and 2 tons per sq ft for samples both parallel and perpendicular to the plane of bedding.

Swelling Determined from Building Deflection Data.—Subsequently, in Section 16, a value of 1.6 tons per sq ft is reported in this field.

12. Summary of Observations on Swelling Pressure

Estimates of the value of the swelling pressure show it to be of the order of 1.75 tons per sq ft. It is the pressure that must be exerted to prevent a free surface of the soil from swelling when in contact with water. As a building prevents uniformity of moisture content under the building, differential vertical movement must occur if the loading does not exceed about 1.75 tons per sq ft.

This fact, allowing for supercharge, offers an explanation for the results reported in Section 7, in which it was shown that for loadings in excess of 1 ton per sq ft there were few, if any, cracks in the superstructure. It also explains the results reported in Section 16 showing deflection curves to change from dome shape to dish shape for loadings in excess of 1.75 tons per sq ft.

13. Classification of Building Cracks and Movements

After eliminating the possibility of building cracks being due to earthquake shocks, the cracks in brick buildings were divided into six main types: (1) Horizontal cracks in the exterior corners of main buildings; (2) diagonal cracks from the foundation; (3) diagonal cracks in the exterior arched walls, supported by pillars; (4) vertical and diagonal tension cracks and shear tension cracks; (5) vertical tension or diagonal shear tension cracks in the bottoms of long and short walls; and (6) vertical cracks in the wall bottoms due to lateral movement.

Type 1.—Horizontal cracks in the exterior corners of main buildings usually occurred about 2 ft above ground level, suggesting that the corners were suspended in midair or that the foundations had sunk relative to the wall. In a block of thirteen single-story cottages constructed at the Agricultural College, these cracks were found in the east and west short exterior walls and occurred in nineteen of the fifty-two corners. In three instances (all in east walls) the cracks extended for the full length of the wall.

Type 2.—Diagonal cracks from the foundation, being often associated with type 1, suggested a local upward movement at the crack focus, resulting (as a secondary effect) in cracks of type 1. Of the twenty-six exterior east and west short walls of the foregoing thirteen buildings these cracks appeared in nineteen. They consisted of a diagonal crack that began at the northeast and northwest corners of the main building, at the junction of the main building

and the outhouses, and radiated diagonally outward at 45° sometimes in one direction and sometimes in both directions. In some cases the crack in the main building wall stopped at the window corner (see Fig. 7), and in others it extended above the opposite corner of the window. In only five walls did the crack reach the top of the wall. It is significant that at point A, Fig. 7, there is a downtake water spout and drain (generally defective) in every wall. Furthermore, similar diagonal cracks were found in many of the return walls.

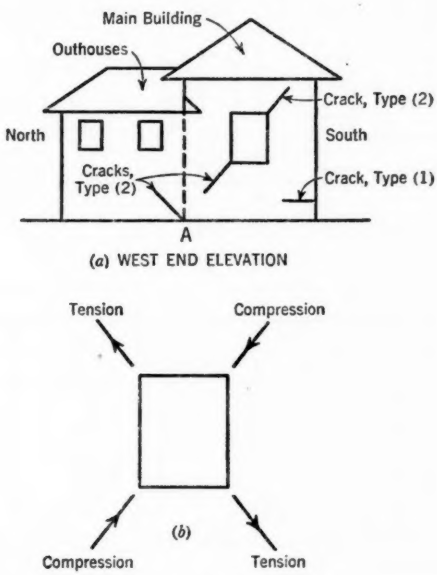


FIG. 7

Type 3.—In the exterior arched walls, supported by pillars, diagonal cracks appeared as the result of relative vertical and horizontal movement. The light arches over the exterior end and exterior intermediate pillars show frequent diagonal cracks. They suggest relative upward and (in an end pier) horizontal movement. The reverse type of diagonal crack is also found suggesting downward movement and, in the end piers, horizontal movement.

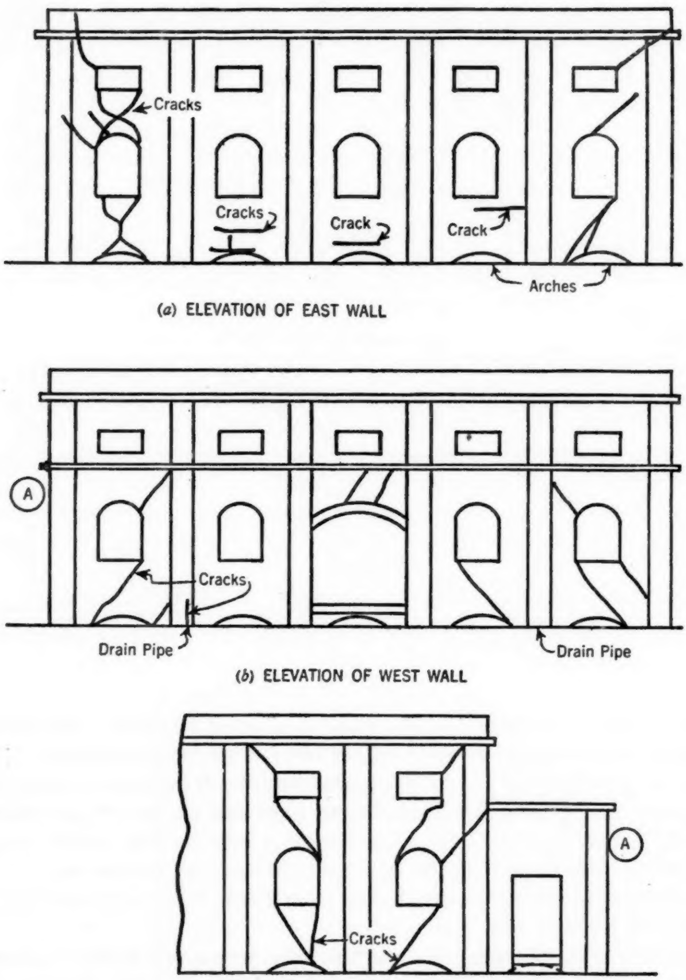
These two types usually extend only from 2 ft to 3 ft and do not necessarily extend to the top of a low wall.

Sometimes both types occur close together, or superimposed, as illustrated in Fig. 8. (In Fig. 8(*c*), note the diagonal cracks in opposite directions.) Repeated distortion (but not settlement) could account for this combination.

Type 4.—Vertical and diagonal tension cracks (in relatively low walls) and shear tension cracks (in relatively high walls) in the tops of long walls occur as the result of relative upward movement of the center of the wall. These are the most common cracks in long walls. They are accompanied by hogbacking, which throws the short end walls out of plumb (see Figs. 1 and 9). A variation of this characteristic consists of a crack that begins as a top tension

crack extending for a few feet down the wall. The crack is then extended and enlarged by horizontal movement.

Type 5.—Vertical tension or diagonal shear tension cracks in the bottoms of long and short walls are due to forces the reverse of which cause the cracks of



(a) ELEVATION OF EAST WALL

(b) ELEVATION OF WEST WALL

(c) PART ELEVATION OF NORTH WALL WITH WEST END PROJECTION

FIG. 8.—OFFICE OF DEPUTY DIRECTOR OF AGRICULTURAL COLLEGE, MANDALAY, BURMA

type 4. They appear in both long external walls and internal cross walls (see Fig. 1). In some instances both types 4 and 5 were found, as shown in Fig. 9. In short walls diagonal cracks caused by shear are found at the junction of the wall with the end walls running at right angles (see Figs. 1(*d*) to 1(*g*)).

Type 6.—Vertical cracks in wall bottoms from lateral movement, occurring close to the end of the wall, appear to be caused by horizontal swelling of the soil enclosed by the exterior foundations.

14. Further Examples of Building Cracks

Cracks in Brickwork Over Arched Foundations.—The best example of cracks in brickwork over arched foundations was found in the walls of the condemned office of the duputy director of agriculture. The walls were riven with cracks and the building appeared to have opened out both at the top and the bottom, as shown in Fig. 8.

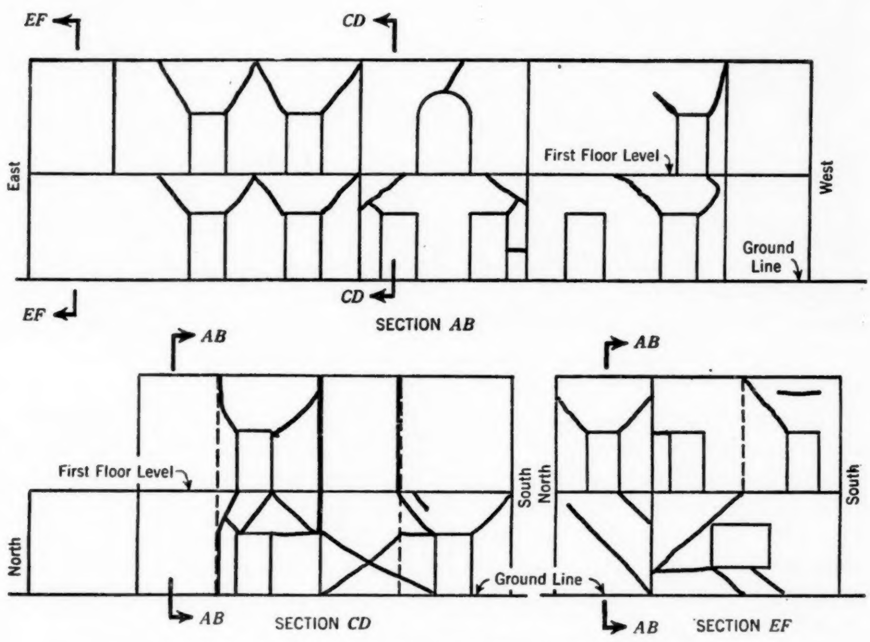


FIG. 9.—MANAGER'S QUARTERS, MANDALAY, BURMA

Vertical Cracks in Brick Water Tank Walls.—A number of small brick water tanks constructed below ground level was examined. These had invariably failed by the side walls being pushed inward, resulting in a wide, vertical central crack in each wall, and in the flooring being pushed upward.

Cracks in and Damage to, Bricknogging Buildings.—These buildings show few cracks and those all of one type. They are mainly confined to the lowest panels and consist of a diagonal crack, at 45°, extending upward from where the post meets its foundation. The floors are uneven, and levels on post footings indicate a wave formation.

Damage to Timber Building on Stilts.—In the "Introduction" reference has been made to the wave deflection of buildings. It occurs in brick buildings but less pronouncedly (see Fig. 2). Because of their elasticity and isolated

footings, timber buildings furnish extreme examples of wave deflection and give a new conception of the difficulties met with in the design of local buildings, especially those of brick. The elevation of barracks (about 180 ft long) shows a distinct and consistent waved formation as illustrated by Fig. 10, facing the south and east walls. The lowest post is not the end one, but a post about 20 ft from either end. The most pronounced dip is still at the west end of the south wall. The highest posts of the south line are the end posts and those about 50 ft from each end. In the intervening 80 ft the wave is not generally quite so pronounced.

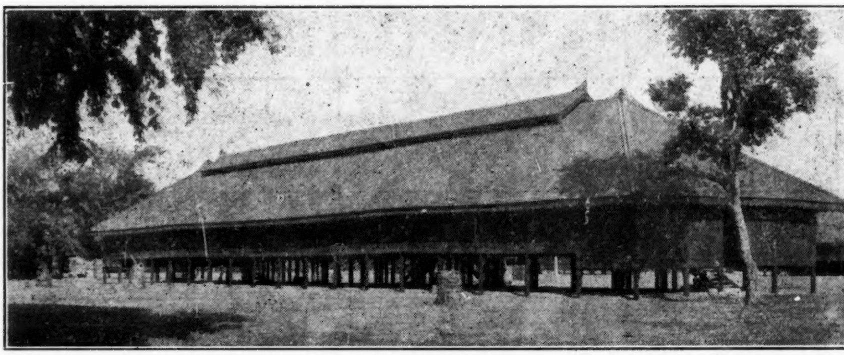


FIG. 10.—WAVE FORM OF SETTLEMENT, TIMBER BUILDING ON STILTS, 180 FT LONG

Damage to Buildings on Pile Foundations.—The quarters for the eight provincial assistants at the Agricultural College rested on 10-in.-square concrete piles, 12 ft long, and those for the three imperial assistants were 15-in.-square concrete piles, 12 ft long. The former carried a maximum load of 7 tons and the latter, 11 tons. When examined in 1935, five of the former were cracked, one of them severely. The main cracks were of type 4 and the partition walls showed horizontal cracks. In the latter buildings several minor diagonal cracks were visible over the piles.

Damage to Buildings on Raft Foundations.—Only one attempt at a raft-type design has been made in Mandalay and that was for the seed laboratory at the Agricultural College. When examined in 1935, many cracks of type 4 were found. Although the failure of this design scarcely merits the condemnation of a good raft design, it furnishes the evidence that, instead of transmitting load to the soil, the soil was loading the slab and, in doing so, the slab had to span lengthwise instead of crosswise, with the result that the raft cracked in a direction perpendicular to its major axis.

Damage to Buildings on Arch Foundations.—Light culvert arches very soon develop cracks and, if not repaired, soon become derelict, as may be seen in almost any part of the dry zone of Burma. Whereas foundations transmitting loadings as great as 1 ton per sq ft crack badly, arches transmitting more than 2 tons per sq ft show little distress.

15. Summary of Observations on Building Damage

In general, the foregoing study of building damage shows it is caused by a combination of vertical and horizontal movements and strongly suggests that the cracks arise out of the peculiar deflection curves assumed by the structures. For such cracks to occur the deflections must be relatively great; they must occur in a time interval insufficient to mobilize the full strength of the structure; or the induced stresses, increased over the theoretical by waving, become too severe for the design. As a further alternative, the foregoing possibilities may be compounded and, in addition, the wave may possibly change during the year.

16. Deflection Curves for Building on Mandalay Soils

Basic Structural Deflection Curves.—In the preceding sections, the more visible evidence of structural cracks was examined and the cracks were classified as explainable by vertical and horizontal movement, or by a combination of the two. In this section the basic deflection curves, with their secondary modifications, are discussed in relation to observations on soils in Mandalay.

The normal basic settlement curve for a flexible rectangular structure, transmitting a uniform pressure to a horizontally homogeneous cohesive soil, is dish shaped. This curve may not be uniform because of the lack of horizontal homogenity of the soil; but, unless any change in the horizontal profile is very marked, the normal basic line remains dominant.

In Mandalay, buildings—mostly on strip or isolated footings—are flexible and the soil is a siliceous, cohesive clay of reasonably uniform homogenity. Therefore, the basic curve should be dish shaped; yet, the normal dish curve for buildings transmitting less than 1.5 tons per sq ft was not found. Although the soil is cohesive, the curves for some seventy buildings resembled those for a flexible structure placed on the surface of a shallow stratum of cohesionless sand[17]—that is, they were fundamentally dome shaped. It follows, therefore, that most of the basic deflection curves cannot result from the Boussinesq distribution of stress.

Actually the basic dome curve, although dominant, is never uniform, especially at the ends. It shows secondary interference which is probably not connected with any change in the horizontal homogeneity of the soil material.

Estimation of Maximum Settlement.—Although no particular significance can be attached to theoretically computed settlements, when conditions do not lend themselves to theoretical treatment, estimates based on the site loading test, the "perimeter-shear" theory advanced by W. S. Housel, M. ASCE, and the Boussinesq and consolidation theories were made for the maximum settlement that might have occurred under the office of the severely damaged Mandalay Race Club.

From the computations the maximum settlement should have occurred under the center of the building, and should have been of the order of 1.32 in. maximum, whereas the maximum relative deflection occurred at one end and was in excess of 2.4 in.

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[17] "The Actual Factor of Safety in Foundations," by Charles Terzaghi, *Structural Engineer*, March, 1935.

Sinusoidal Curves of External Wall Deflections.—A study of the form of the deflection curves for short buildings is of interest when compared with the curves for long rectangular timber buildings. Let AG, Fig. 11, represent the curve of a building 180 ft long, as illustrated by Fig. 10, taking the form ABCDEFG, common to all such buildings, where the section CE is similar to, but not so pronounced as, AC or EG. The bricknogging office is 120 ft long, loaded 0.5 ton per sq ft, and its curve (see Fig. 12) can be represented by AL (112½ ft, in Fig. 11) in which peak C is dominant as compared with peak D. As indicated in Fig. 12(*a*), levels were plotted with the northwest corner post footing at an assumed datum elevation of 100.000. There is no justification for this assumption, but it enables a comparison to be made of the several curve shapes. Actually, it is quite clear that in this building there is no area of constant elevation. Figs. 12(*a*) and 12(*b*) show that, with the datum used, the maximum movement in the exterior lines occurs at the center, whereas, for the interior lines, it occurs at the third points. The maximum movements between the curves in Figs. 12(*a*) and 12(*b*) are 1.5 in., occurring, relatively, at the center of the veranda line (see Fig. 12(*c*)). The maximum deflection recorded was $d = 2.88$ in. The length of the chemical laboratory is 100 ft and its cracks and outline are fairly well represented by BE (112½ ft in Fig. 11) in which peak C is dominant as compared with peak D. The race club office was 62 ft long and its curve (see Fig. 2) can be represented by BD, Fig. 11, which is 67.5 ft. The electrical substation is 48 ft long and its curve can be represented by AC (Fig. 11), 45 ft long.

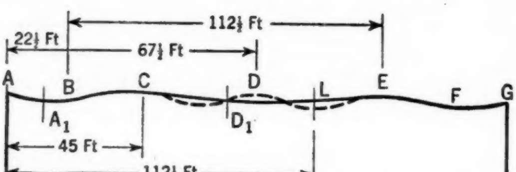


FIG. 11

The foregoing examples of simple, light, rectangular buildings suggested a common fundamental sinusoidal deflection curve of a constant wave length for all buildings and supported the belief in soil buckling. There is no suggestion, however, of a common wave length in the curves for more irregular or heavier buildings. The phenomenon appears to apply only to rectangular buildings transmitting loadings of less than 1.5 tons per sq ft.

Connection Between Basic Deflection Curve, Deflection, and Loading for All-Brick Buildings.—Deflection curves, for the limited number of brick buildings for which curves could be obtained, show variations in maximum relative vertical movement from +4.68 in. (dome) to −1.92 in. (dish) and in maximum deflection from +2.88 in. (dome) to −1.44 in. (dish), for a loading range varying between 0.25 ton per sq ft and 2.1 tons per sq ft. In most curves there is secondary interference especially at the ends. The deflection results are plotted in Fig. 13 and (with one exception to be analyzed, subsequently) show a straight-line relationship between the loading P and the ratio of deflection, d, to the length of building, L.

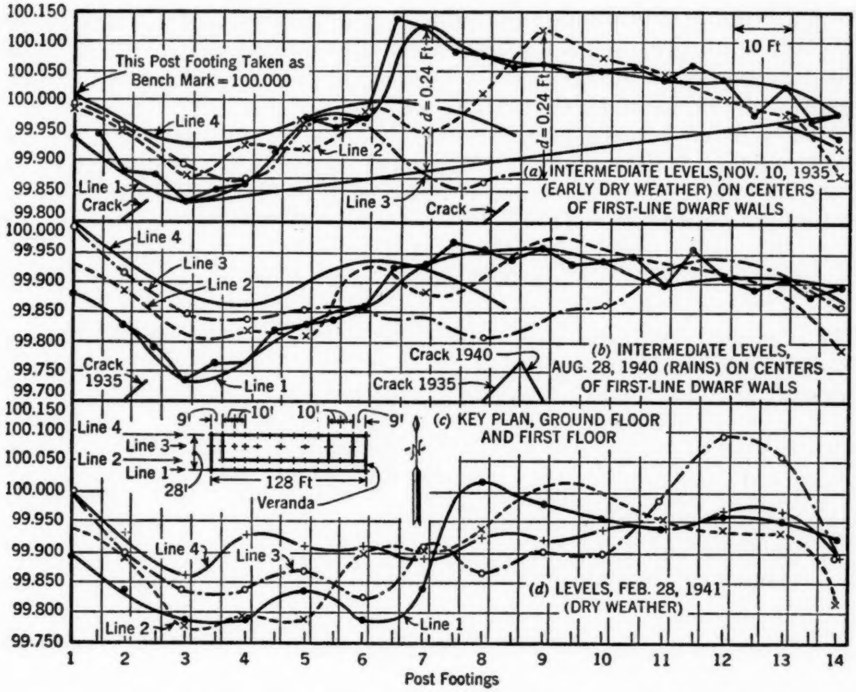


FIG. 12.—FOUNDATION LEVELS ON OFFICE BUILDING, MANDALAY, BURMA

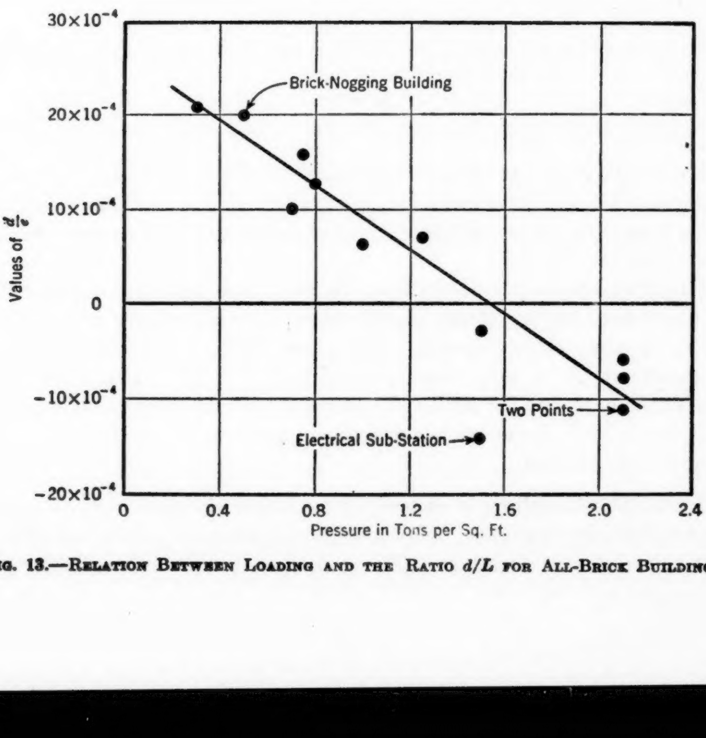


FIG. 13.—RELATION BETWEEN LOADING AND THE RATIO d/L FOR ALL-BRICK BUILDINGS

This ratio (d/L) becomes zero for a pressure just less than 1.5 tons per sq ft—that is, when the soil is loaded to about 1.75 tons per sq ft, at which value the deflection curve changes from a dome shape to a dish shape. This value agrees with that found for the swelling pressure in Section 12.

The relation between transmitted loading and relative deflection for all-brick buildings is determined as follows: Let—d be the relative deflection, in feet; L be the length of structure, in feet; and P be the transmitted pressure, in tons per square foot. Then

$$\frac{d}{L} = 0.00266 - 0.00171\,P \qquad (2)$$

The equation of the laboratory determined curve of swelling pressure versus swelling : compression is expressed by—for the plotted range of $P = 0.5$ ton per sq ft to $P = 2.0$ tons per sq ft—

$$S = 0.0063 - 0.0039\,P \qquad (3)$$

in which S is the test swelling or compression, in feet, and P is the pressure release on swelling. The length of the specimen is 0.87 in.

Combining Eqs. 2 and 3, eliminating P, and solving for S, these two equations give an approximate interrelationship for a similar loading of

$$S = 2.3\,\frac{d}{L} \qquad (4a)$$

and the (test) osmotic range strain is expressed as

$$\epsilon = 32\,\frac{d}{L} \qquad (4b)$$

Eq. 2 can thus be expressed, for any load and proportional osmotic strain, as

$$\frac{d}{L} = \epsilon\,\frac{1.55 - P}{1.55} \times \frac{1}{32} \qquad (5)$$

in which $\epsilon = 0.085$ against the average value of 0.087 for the agricultural shrinkage coefficient for five sites between the depths of 4 ft and 12 ft. Curves of both Eqs. 2 and 3 gave an effective swelling pressure of 1.6 tons per sq ft (1.55 and 1.65, respectively).

Factors Responsible for Deflection.—With dome and sinusoidal deflection curves unexplainable by stress distribution, it is necessary to consider the possibility of upward movement. The phenomena which might lead to upward movement are briefly discussed. They are (1) vertical soil swelling, (2) horizontal external forces, (3) horizontal soil shrinkage, (4) soil circulation, (5) local clay areas with pronounced sodium characteristics, and (6) thixotropy or dilatation.

(1) Vertical soil swelling is caused by moisture changes such as may occur at the end of the hot weather, or by differential changes arising out of the time lag in the flow of rain moisture. These changes are accompanied by changes

in pressure between the foundations and the soil and would be expected to lead to movement if the external loading were less than the swelling pressure of 1.75 tons per sq ft.

(2) Horizontal external forces on the foundation walls due to external soil swelling may produce a dome-shaped curve and force the end walls out of plumb.

(3) Through adhesion between the soil and the foundation, horizontal soil shrinkage under a foundation may cause central upward movement in a light building. Approximate calculations show that a vertical loading of about 1 ton per sq ft is necessary to overcome this friction force.

(4) During the dry weather, when soil cracks appear, soil and other matter fall into crevices and the process is aided by various external agencies. When the rains start, water-borne material is also deposited. If a building is founded on an artificial sand cushion, some of the sand must enter any soil cracks that may occur. This circulating material may cause a local settlement but its presence when the soil begins to swell must tend, by conjugate pressure, toward an upward or wave movement of the soil. On the whole, this would be expected to occur, particularly at the ends of buildings. Proof of this soil circulation was noted during borings at the Agricultural College (dry area) when lumps of mottled black and yellow clays were occasionally extracted from as low as 16 ft.

(5) Local clay areas with pronounced sodium characteristics may lead to buckling if, for such soils, the suspected difference between horizontal and vertical shrinkage is an actuality.

(6) Thixotropy (the property or phenomenon, exhibited by some gels of becoming fluid when shaken) or dilatation, or a combination of each, may be responsible for some local buckling in the same way as soil circulation.

Evidence of Upward Movement.—Through rather a revolutionary conception, difficult to detect, upward movement follows as a corollary to the pressure release of 1.75 tons per sq ft during swelling over the residual shrinkage range unless the loading exceeds this value. The direct evidence in support of its actuality is as follows:

(*a*) The southwest corners of foundations, although most exposed to alternate drying and wetting, are often higher than the foundations to the east, or even those inside the building.

(*b*) Stone, or concrete, floors appear to settle considerably, especially in the southern half of the buildings. This settlement (if it is a settlement) may be presumed to occur uniformly until a condition of equilibrium be reached. At the end of the rainy season it has been noticed that footings in the flooring around the exterior walls—especially south walls protected by a veranda—indicate relative upward movement accompanied by occasional cracks of type 2 (Section 13). This movement occurs particularly under southerly exterior posts, subject to full weather effects, and, hence, to maximum changes in voids ratio. Therefore, it is considered to be direct evidence of upward movement of the footings (Fig. 14).

(*c*) Dwarf veranda walls between post footings often develop a dome curve with top, central cracking. This type of settlement is considered, principally to be due to the upward movement between the posts caused by soil swelling

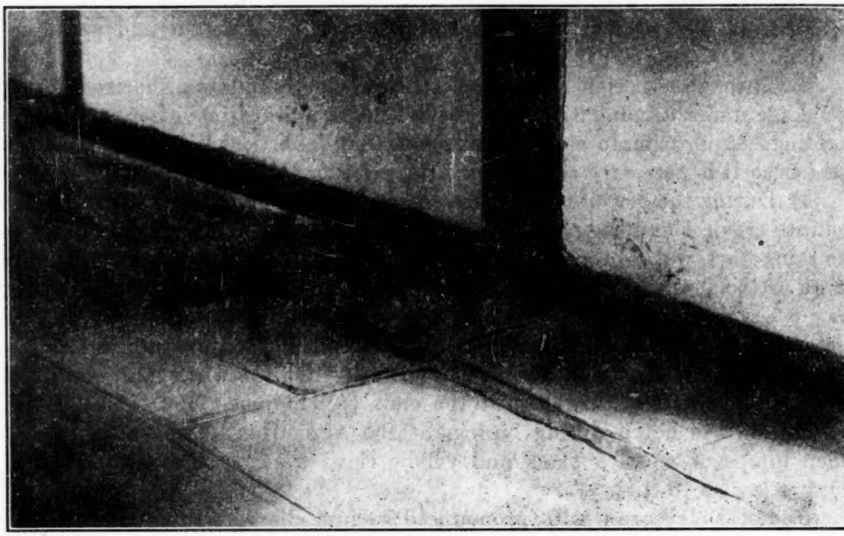


FIG. 14.—STONE OR CONCRETE FLOORS APPEAR TO SETTLE CONSIDERABLY

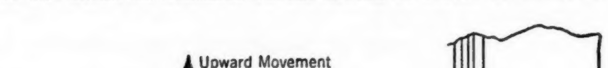


FIG. 15.—UPWARD MOVEMENT OF PORCH FOOTINGS, CIVIL SURGEON'S QUARTERS, YAMETHIN, BURMA

against no appreciable resistance, although it may be due in part to some downward movement of the posts. An example of a reinforced-concrete tie beam, connecting two pile tops falling under the same forces, is described in

Section 11. The deflection curves for veranda posts in Fig. 12 supply an additional example.

(*d*) Cracks, particularly of types 2 and 4 are indicative of upward movement.

(*e*) A test, especially devised to prove the actual occurrence of upward movement and to measure its magnitude, is described in Section 19. The test confirmed the movement. The effect of upward movement is illustrated by Fig. 15 which shows the cracking and movement of a lightly loaded porch way of a house in Yamethin, Burma. To a depth of 4 ft the soil was a stratum of sandy clay loam, overlying a stratum of colloidal black cotton clay to a total depth of 7 ft.

17. Summary of Factors Responsible for Deflection Curves

It is clear that, as the factors productive of normal settlement are relatively insignificant, the secondary phenomena responsible for movement in an expansive soil, whose normal moisture content is confined to the residual shrinkage range of undisturbed soil, must be taken into consideration. Most of the phenomena discussed have been proved to exist and the forces induced by them have been shown to be of appreciable magnitude. The most important of these, applicable to any site, appears to be that of soil swelling involving the conception of upward movement.

18. Cyclical Movements

Movements Detected During Load Test.—A study of the levels taken on the tank which was used for the loading test (Section 7) and on the four indicator pegs with one of the office post footings as datum, showed (from the results over the period from October 14, 1935, to April 2, 1936—that is, from the most saturated period to the most dry period) cyclical variations in level. The level of the tank relative to the bench mark on the post footing, founded on black cotton soil at a depth of 4 ft, showed similar variations in level. In particular, they indicated that toward the end of March, just before the lower horizons begin to take up nonrain moisture (see Fig. 4), the relative difference in level between the loaded test area and the post footings changed by 0.48 in., suggesting that the post footing had settled by 0.48 in. Within twenty days this difference had recovered to 0.36 in., which is explainable by the upward arrival of moisture under the test load, resulting in some upward movement of the test load relative to the post footing.

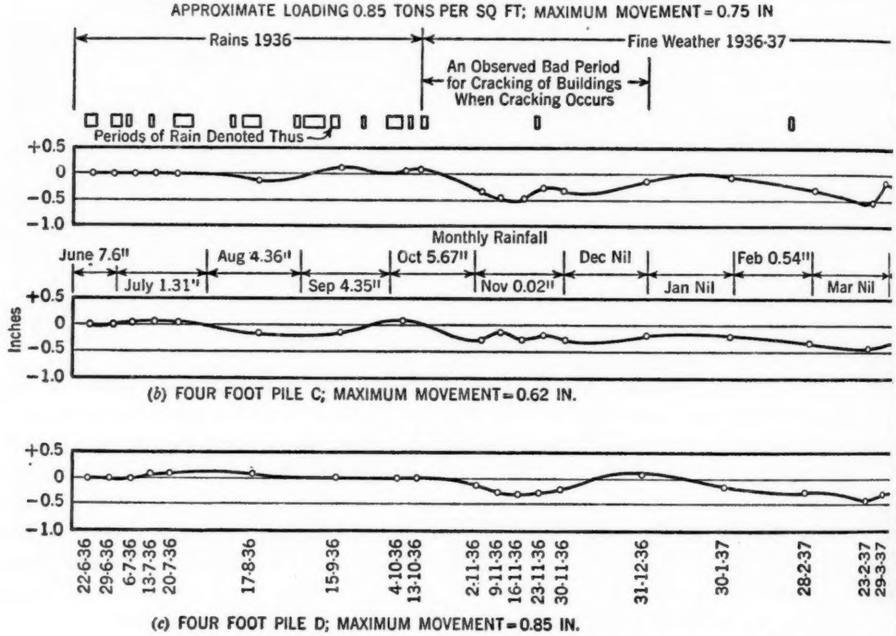
The apparatus was not considered suitable to permit drawing any definite conclusions and the test described in Section 19 was devised to check this phenomenon.

19. Cyclical Movement Demonstrated by Pile Test

To verify the upward movement of buildings, use was made of the seasoned "pyinkado" (hard timber), 8 in. by 8 in., pile driven to refusal at a depth of 21.5 ft. Two short piles, each 4 ft long, were driven in line with the 21.5-ft pile and spaced at 11-ft and 5.5-ft centers. Levels were taken periodically on each pile top, flush with the ground, and on one corner of the foundation.

The possibility of the long pile varying in length with possible moisture changes in the surrounding soil was considered, and it was found, for the various conditions examined, that any changes in pile length were unappreciable compared with movements actually measured. The variations in the levels of the short piles and in the godown footing, relative to the long pile as datum, are shown in Fig. 16. They are of a cyclic nature. Comparison of these curves with those for variation in moisture:saturation capacity, in Fig. 4, shows close agreement although the two periods observed are not entirely identical.

(*a*) COLUMN FOOTING A, FOUNDATION LEVEL, 5.5 FT BELOW GROUND LEVEL; APPROXIMATE LOADING 0.85 TONS PER SQ FT; MAXIMUM MOVEMENT = 0.75 IN



(*c*) FOUR FOOT PILE D; MAXIMUM MOVEMENT = 0.85 IN.

FIG. 16.—VARIATIONS IN LEVELS OF FOOTINGS AND 8 IN. BY 8 IN.,

The results, over a period of eighteen months, shown in Fig. 16, illustrate:

1. The movement in a building, transmitting less than 1.5 tons per sq ft, during the course of a year;

2. The uneven movement of the soil surface, as indicated by the short pile curves, simulating buckling and suggesting a moving wave; and

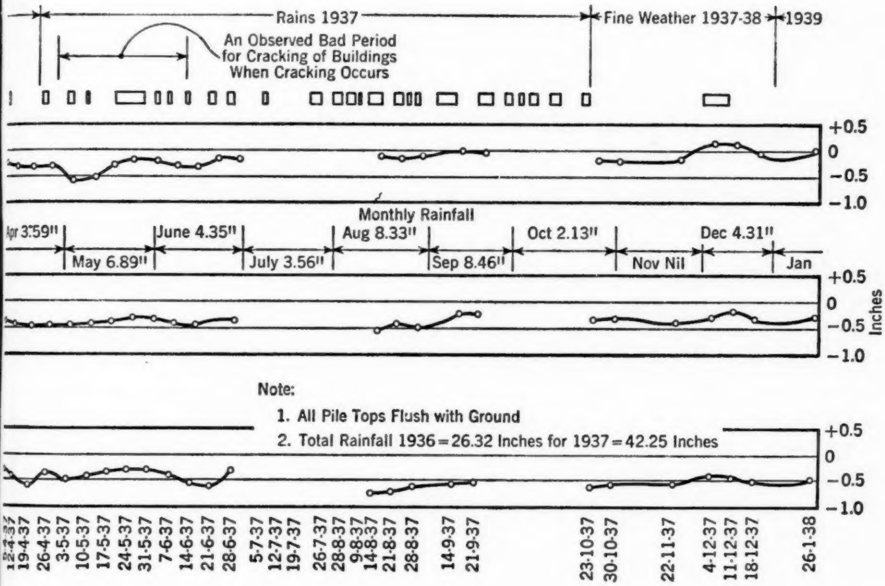
3. The accumulative effect of item 2, illustrated by the difference in levels of piles C and D at the end of the test period.

20. THE PHENOMENA

Old Theory Reviewed.—The main feature of the old theory of the cause of cracking of buildings in Mandalay was that the soil was of low carrying capac-

ity, especially during the rains. The fact that kyatti soils turned rapidly into a soapy fluid in the presence of water was accepted as supporting evidence. This observation is correct only when applied to disturbed soils and dynamic water. In Mandalay the high dispersion factor, caused by the replaceable sodium, certainly accentuates this condition, but clays under foundations are neither fully disturbed nor subject to the eroding action of free dynamic water.

It has been shown in Section 7 that the soil will readily carry a pressure of 2 tons per sq ft under very severe moisture conditions and that even under



…F SHORT PILES REFERRED TO A TIMBER PILE,
…ND 21 FT LONG

these severe conditions there was no continuous plastic yield. The actual yield was small and instantaneous. The consolidation test supported the site test and showed that the soil is normally in a condition corresponding to a carrying capacity, when saturated, of nearly 6 tons per sq ft. (This pressure was reached during a loading test at Yamethin when the apparatus—not the soil—failed.) Observation supports the high carrying capacity of the soil in test observations and in the comparative absence of cracks in the heavier buildings.

The presence of building cracks close to a leaky drain has been quoted as evidence in favor of judging the carrying capacity to be low. Examination shows that these cracks result from a local upward movement over the seepage area and not from downward movement due to soil yield. These facts dispose

of the theory accrediting the occurrence of cracks in buildings to a low bearing value.

The Cause of Dome-Shaped Basic Deflection Curves for Pressures Less than 1.5 Tons per Sq Ft.—It has also been shown that a load of at least 2 tons per sq ft can be carried safely with little settlement and that for loadings greater than 1.5 tons per sq ft little cracking appeared in buildings. The structural, basic deflection curve has been shown, generally, to change from the usual dome to the expected dish shape for loading in excess of that corresponding to the swelling pressure of 1.75 tons per sq ft. This suggests that the accepted conception of a dish-shaped deflection curve applying to structures sited on a horizontally homogeneous cohesive soil requires amplification to yield a more general law introducing the moisture-content factor. For the clay soils examined, the form depends on the ratio of loading to active swelling pressure—that is, it is apparently a function of the moisture fluctuation range on the undisturbed shrinkage curve.

The explanation offered for the dome curve in the case of brick buildings transmitting less than 1.75 tons per sq ft, in lieu of the theoretical dish curve, is based not on consolidation but on differential moisture changes brought about by natural drying and wetting over a moisture range that is represented, almost entirely, by the residual shrinkage range of the undisturbed shrinkage curve where during wetting small voids changes correspond to high internal pressure changes. These moisture changes occur to depths of at least 12 ft.

The process involved in the dome curve formation is considered to depend on: (1) The original and final moisture content of a core of soil under the central part of the structure; and (2) the high alternating volumetric changes occurring outside the central core—that is, against and under the exterior foundations.

Assuming excavation to be done during the dry weather (which is usual) and that, as so often happens, the foundations are open for some time, then the exposed foundation soil will dry out to a moisture content of about 6% and shrink to a voids ratio of, about, 0.567—corresponding to a loading of about 6 tons per sq ft.

After the first heavy rain the soil outside the building and immediately under the exterior foundations will swell to a voids ratio of about 0.705, where unloaded and exposed, and to about 0.63, when the loadings are 1 ton per sq ft.

In addition to a general tendency toward upward movement, this swelling must cause an upward movement of the building ends relative to the building center because of the forces on the end walls, the flexibility of the building in the direction of its major axis, and the time lag in moisture flow. The condition is similar to a beam held at the center and loaded upward at its ends; and it tends to produce a dish curve, but actual movements must depend upon the moisture content of the central core at the time of construction. However, because of rapid drying, this initial upward movement is of short duration.

After a series of such movements, the central core will (in time) absorb moisture to some maximum value which will probably occur during the dry weather and by preventing evaporation, this maximum value will be retained more or less permanently. This action will result in an ultimate central support through the cross walls and the inertia of the structure. The building then acts

as a centrally supported, double cantilever and the deflection curve is in the the process of being changed from a dish shape to a dome shape. During the same period there is a general tendency for the exterior walls to settle to their original positions. This settlement will be most pronounced at the building ends because of the weight of the end walls, the flexibility in the direction of the major axis, and the support of the central core. The result is the deflection along the major axis and the formation of a dome-shaped curve.

During this change-over period, any late rains (as may occur in November or December) may cause some incremental additional end settlement through the local breaking down of the soil structure, which results in very small accumulative amounts of plastic yield.

As evaporation and temperature changes are a minimum for the central core, it is considered that its moisture intake will be retained, in general, and that any later fluctuations will be small compared with those under the external foundations. Consequently, the central core is left at a higher level than when constructed during the dry weather, and thus it acts as a final support through the inertia of the building.

The upward movement of the central core does not cause relative movement in the internal walls sufficient to cause severe cracking. If it did, the cracks illustrated in Fig. 1 would have been in the opposite direction. Since there must be a tendency for this cycle to be repeated yearly, it would appear that, through possible accumulative incremental end sinkage, the dome curve becomes somewhat more pronounced with time.

Alternatives and Exceptions.—In the event of the central core being saturated during construction, variations in the aforementioned cycle are possible. Each will depend upon the relative moisture changes and such secondary causes as drainage. Thus it is conceivable that the curve will pass straight into the dome shape, or, even, that the final curve will be dish shaped. Furthermore, it seems possible that, when construction is followed by a number of wet years and then by a number of dry years, an originally developed dish curve may change into a dome-shaped curve after several cycles, although in such an example the final central level will probably be lower than when constructed. Should the soil be fully saturated continuously, the swelling pressure would be negligible and a dish curve appears inevitable.

Reference has been made in Section 16 to an exception to the straight-line relationship between the loading and relative deflection. This is the Agricultural College electrical substation, and its pronounced dish curve is a result of the unusual local conditions under which one of the foregoing alternatives has developed. This building is situated on low land, waterlogged for much of the year, and within 50 ft of a main irrigation distributary. Under such conditions a pronounced and natural dish curve may be expected to develop.

21. The Effect of Flexibility on the Basic Curve for Loadings of Less Than 1.5 Tons Per Sq Ft

Bricknogging Buildings and Timber Buildings on Isolated Footings, But Not on Stilts.—From the discussion on the formation of the basic curve for all-brick buildings it might be expected that nogging buildings would exhibit

the same deflection curve although not in such a pronounced form because of the discontinuity between the interior and exterior foundations. The lines of interior footings should also follow the general form in a lesser degree, with a suitable lag. Levels are in accord with this expectation.

Timber Buildings on Stilts.—For such buildings, at which moisture fluctuations may occur over the entire area, the basic curve should be the least evident and deflection might be considered to arise principally from tertiary effects. However, as the moisture fluctuations of the central core must be less than those under the exterior footings the theory suggests that the major axis should be higher than the exterior foundation lines. Barracks show sinusoidal deflection curves but no basic dish or dome curve. The end elevations of the barracks, shown in Fig. 10 and applicable to all barracks, illustrate that this does occur in the field.

The theory evolved by the findings reported in this paper explains the less pronounced basic curve for buildings on isolated footings and the almost complete absence of any basic curve for buildings on stilts. In general, it supplies a reason for the tertiary effects increasing with the flexibility of the building as it changes from an all-brick building to a timber building on stilts. By the proposed theory, rafts and piles (unless fixed deep in a constant volume horizon, or kept at a constant volume by the superimposed loading) should move in the same way as a brick building on strip footings. This again has been found to be so.

22. The Cause of the Sinusoidal Curves of Deflection

In Section 16 reference was made to the fact that the basic dome curve, for loadings less than 1.5 tons per sq ft, is subject to tertiary effects leading to a sinusoidal curve. In Section 21, it was demonstrated that the degree of interference depended not only on the loading but also on the flexibility and the type of design.

Since the possibility of soil buckling arising out of unequal horizontal and vertical shrinkage should be excluded (for the present) as a generally applicable major cause of wave action, the only explanation appears to lie in the accumulative effects of the factors producing upward and downward movement (see Section 16), and in the inertia of the structure. Of these possibilities the only ones that are generally pertinent are differential movement due to moisture changes and soil circulation. For a long brick building, both of these factors would be expected to be most apparent at the ends where the interference is most noticeable.

23. The Stability of the Deflection Curves

Movement is not solely confined to one direction to form one of the basic curves, but may alternate as demonstrated by the coexistence of cracks from both upward and downward movement. As explained in Section 20, the aforementioned dish curve is considered to form for low loads, if it ever forms, during or soon after construction and may in time change to a dome curve. Whether further changes in the basic curves may occur is not known but it is

significant that, when the Agricultural College was examined in 1935, it was recorded that the building levels showed slight hogbacking, whereas when it was leveled on April 1, 1940, it was dish shaped. Further releveling on February 28, 1941, indicated a distinct change in the curve, suggesting that any change-over from dish curve to dome curve, or vice versa, may apply particularly when the loading is in the region of the swelling pressure, as is true of this building.

Whether the positions of the crests of the sinusoidal curves are subject to lateral movement has not been proved. Perhaps the inertia of the building prevents this while permitting accumulative differential movement in other respects. It is clear, however, that cyclic vertical and horizontal movements occur, and, from Section 19, that some minor lateral movement in the crests should occur. The releveling of the office building in Fig. 12, after five years, showed the level lines to be similar regarding positions of crests and hollows, although there were considerable displacements between the level lines and differences in the relative heights of the crests. Further relevelings at this building and at the Provinical Police Training School showed similar relative movements occurring within the shorter periods of one month and six months.

Hence, once a condition of complete, or partial, equilibrium of moisture content of the central core has been established, it appears that the main features of the deflection curve are maintained; but the curves, nevertheless are subject to cyclical variations.

24. Acknowledgment

The material for the preparation of this paper is from a report submitted by the writer to the Government of Burma in 1945, entitled "The Phenomena Involved in the Failure and Partial Failure of Buildings Constructed on the Desiccated Alkaline Soils of the Dry Zone of Burma." For constructive assistance, the writer is indebted to: The director and staff of the Agricultural College, Mandalay; Mr. Robinson, University of North Wales, Bangor; Mr. Clark, University of Illinois, Urbana; R. E. Stradling, director, Building Research Station, Watford; R. Kenworthy-Schofield, Rothamsted Experimental Station, Harpenden, Hertford, England; Hans F. Winterkorn, Assoc. M. ASCE, Princeton University, Princeton, N. J.; L. F. Cooling, Building Research Station, Watford; and G. R. Clarke, School of Rural Economy, Oxford University, Oxford, England.

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PAPERS

CORRECTION OF RESERVOIR LEAKAGE AT GREAT FALLS DAM

By A. H. Weber,[1] Assoc. M. ASCE

Synopsis

Leakage of more than 450 cu ft per sec from the Great Falls Reservoir (in White and Warren counties in Tennessee on the Caney Fork River, a tributary of the Cumberland River), acquired by the Tennessee Valley Authority (TVA) from a private utility company in 1939, was reduced 98% by injecting hot asphalt and cement into holes drilled to intercept the water-bearing rock fissures. A total of 608 holes was drilled and grouted along a cutoff line almost a mile long.

The leakage, which had been steadily increasing since the dam was raised in 1925, was a threat to the useful life of the Great Falls hydroelectric project. Before correction, the leakage represented about 14% of the water used by the two hydraulic turbines when in full operation.

This paper discusses the causes of leakage, and the limited usage of cement and asphalt in a grouting program designed to correct the leakage at a justified cost.

Introduction

Leakage from the Great Falls Reservoir increased progressively until a flow of more than 450 cu ft per sec at full reservoir was recorded just before corrective work was initiated in 1945. This leakage equaled approximately 14% of the over-all full-load water use of the two Francis hydraulic turbines installed in the power plant. The electrical energy loss amounted to about 20,000,000 kw-hr annually.

The average annual leakage increase since 1925 was about 23 cu ft per sec. Measurements made by the TVA after 1939 indicated that the average annual increase since 1939 was close to 30 cu ft per sec. Corrective work undertaken in 1945 revealed that the leakage channels were in an early stage of development and that continued acceleration of annual leakage increases could be expected.

Note.—Written comments are invited for publication; the last discussion should be submitted by June 1, 1950.

[1] Constr. Engr., TVA, Chattanooga, Tenn.

The Great Falls hydroelectric project consists of: (1) A gravity-type concrete dam which impounds a "three-fingered" reservoir on the Caney Fork River and on two of its tributaries, the Collins and Rocky rivers; and (2) a powerhouse, 4,000 ft downstream from the dam, which is supplied with water from the reservoir through tunnels and penstocks. The powerhouse is equipped with two generating units that have a maximum discharge of 3,200 cu ft per sec. The project was originally placed in operation in 1916, with a low dam and one generating unit. In 1925 the dam was raised 35 ft to its present level, increasing the height of the reservoir to El. 805. The powerhouse was reconstructed, and the second generating unit was installed.

FIG. 1.—AIR VIEW OF THE GREAT FALLS PROJECT

The location of the powerhouse a considerable distance downstream from the dam is made possible by the topography of the area. The Caney Fork River between the dam and the powerhouse flows through a deep gorge. About midway between the dam and the powerhouse, the river has cut an inner gorge creating a series of falls and cascades, with the result that the river level at the powerhouse is about 75 ft lower than that at the dam. The Collins River enters the Caney Fork immediately above the dam and loops back more or less parallel to the Caney Fork as shown in Fig. 1.

The loop in the alinement of the Collins River creates a single ridge which varies in width from 600 ft to 3,500 ft and separates the lower 2 miles of the reservoir on the Collins River from the gorge of the Caney Fork River down-

stream from the dam. Through this ridge extensive leakage from the reservoir developed, and formed a series of waterfalls and cascades beginning near the river level just below the dam at the left abutment rising to a height of more than 60 ft near the powerhouse. A small amount of leakage also found outlets near the right abutment of the dam.

The formations present in the area are, in ascending order, the New Providence, the Fort Payne, and the Warsaw. The New Providence, a greenish siliceous shale, appears in the Caney Fork in the vicinity of the powerhouse. This formation lies well below the bottom of the reservoir, and its presence has no effect on the leakage problem.

Directly overlying the New Providence formation is the Fort Payne chert. This formation contains two members—the lower about 55 ft thick and the upper about 45 ft thick. The lower member of the Fort Payne is a medium-gray, fine-grained limestone, interbedded with an abundance of tough, dense, dark chert. Because of its highly siliceous nature, the member is resistant and relatively insoluble. The upper member is a medium-gray, fine-grained to medium-grained limestone. A considerable amount of both dense and porous chert is present, but the chert is less abundant than in the lower member. Cavities occur anywhere in the upper Fort Payne formation but they tend to concentrate toward the bottom. Practically all the leakage from the reservoir was through the upper Fort Payne formation.

Overlying the upper member of the Fort Payne formation is the Warsaw shale, a stratum of fissile, bluish-gray calcareous shale. Its depth varies but its usual thickness is about 20 ft. The stratum is relatively insoluble and impervious, and forms an effective barrier to the downward percolation of surface water. Surface water from the dividing ridge forms a system separate and distinct from that of the reservoir leakage. A geologic section plotted from drilling logs is shown in Fig. 2.

The transmission of leakage occurred through channels in the rock obviously developed long before the dam was constructed. Since the leakage strata are covered by the impervious bed of shale (which is entirely capable of protecting the underlying rock from the attacks of surface water), it appears that the channels were formed by river waters, probably in the following manner: As the Collins River cut downward, it passed through the shale and reached the soluble limestone beneath it. Joints and bedding planes permitted slow and minute entrance of the water into the rock; and, as the surface of the near-by Caney Fork River was considerably lower than that of the Collins River, a gradient existed, and the water eventually found its way through the divide. The minute openings were slowly enlarged by solvent action into appreciable channels. As the rivers continued to deepen their gorges, the subterranean channels that had been formed were left above river level and new ones developed at lower elevations. In this manner, a series of entrance passages came into existence, extending vertically from the top of the Fort Payne formation to the present river bed.

When the lake was first formed, the old channels were submerged again, but the residual clay in the channels largely prevented flow of appreciable

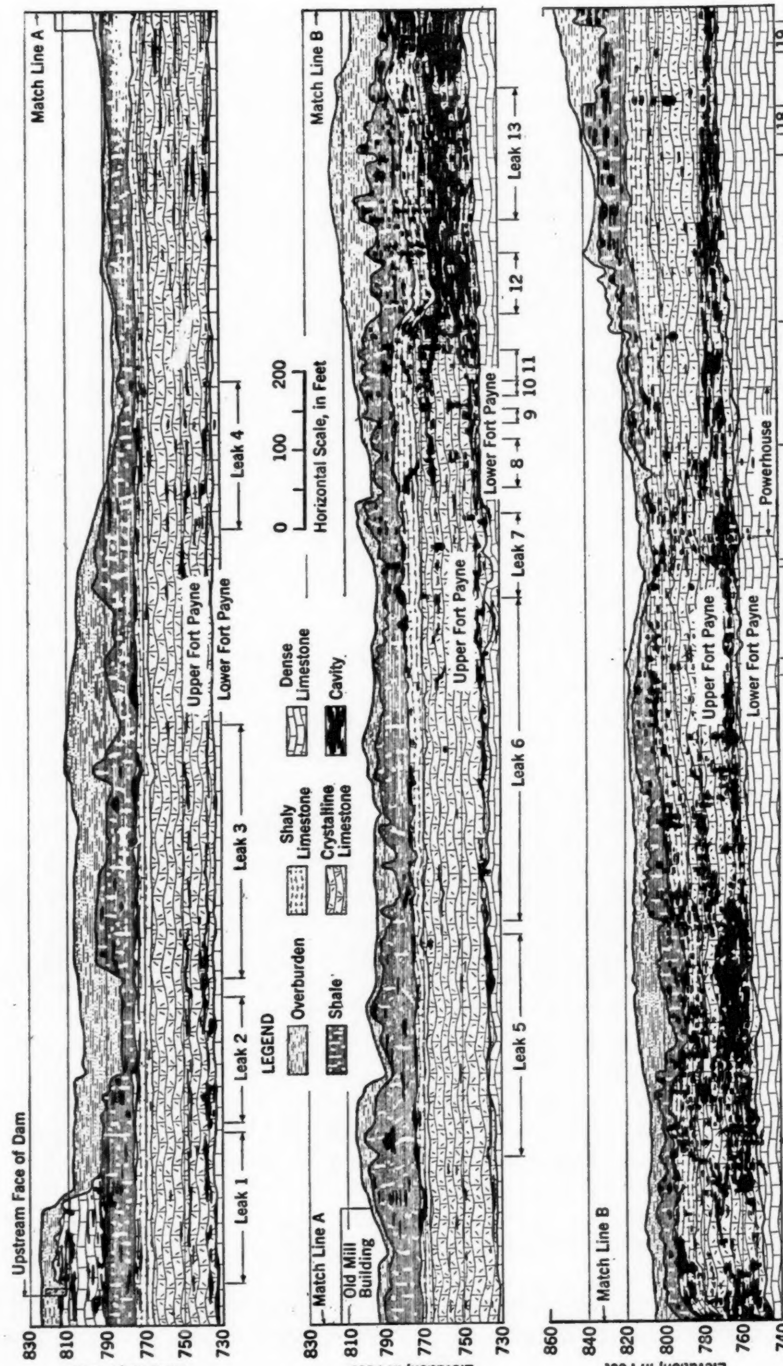


FIG. 2.—GEOLOGIC SECTION ALONG LINE OF GROUT CURTAIN, BETWEEN THE DAM AND THE POWERHOUSE

volumes of water through the passages. In time, however, initial seeps enlarged into flows, as the clay eroded from the channels, with the result that progressively increased leakage found its way from the reservoir.

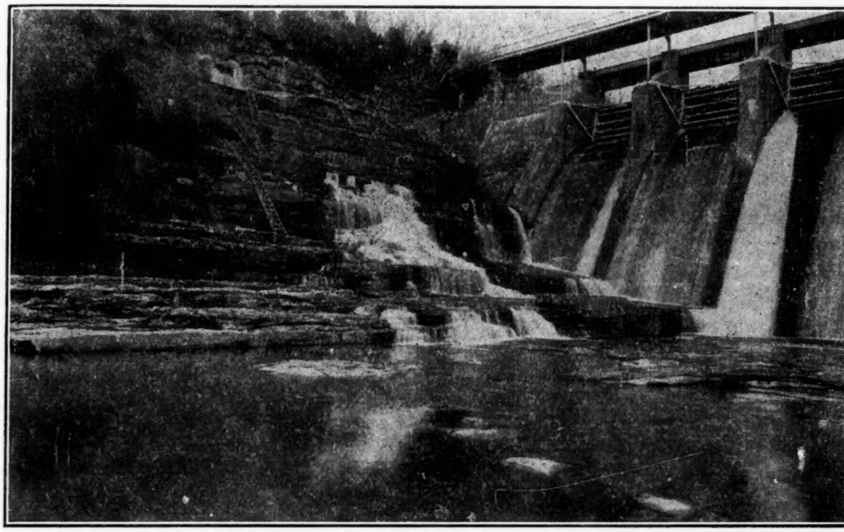


FIG. 3.—LEAKAGE NEAR RIGHT ABUTMENT OF DAM (LEAK NO. 20, LEAKAGE SYSTEM NO 1) BEFORE CORRECTION

FIG. 4.—LEAKAGE NEAR RIGHT ABUTMENT OF DAM (LEAK NO. 20, LEAKAGE SYSTEM NO. 1) AFTER CORRECTION

There were three major areas, each separate and distinct from the others, from which leakage entered the Caney Fork River:

Leakage system No. 1 was on the right bank, and transmitted water around and under the right abutment of the dam. Water entered this system through

inlets close to the dam and flowed under the right abutment. It escaped just below the dam near the contact of the Fort Payne and Warsaw shale as shown in Figs. 3 and 4. At high reservoir stages this leak had a volume of 20 cu ft per sec. It ceased to flow when the reservoir surface fell below El. 768.

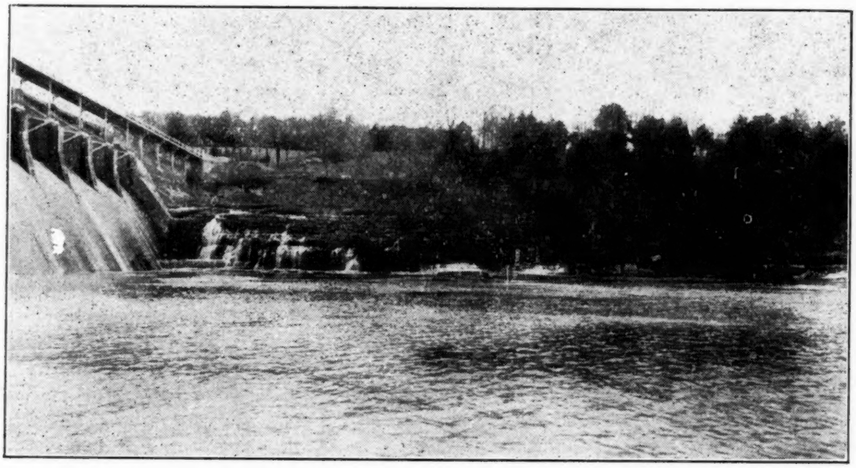


FIG. 5.—LEAKAGE NEAR LEFT ABUTMENT OF DAM (LEAKAGE SYSTEM NO. 2) BEFORE CORRECTION

FIG. 6.—LEAKAGE NEAR LEFT ABUTMENT OF DAM (LEAKAGE SYSTEM NO. 2) AFTER CORRECTION

Leakage system No. 2 was on the left bank of the Caney Fork River. It was supplied by inlets extending from the dam upstream for a distance of more than 5,000 ft. The major part of this leakage was discharged in three falls about 300 ft downstream from the dam. At minimum reservoir elevation this system of leakage discharged 62 cu ft per sec; and, at maximum pool, it discharged 150 cu ft per sec (see Figs. 5 and 6).

Leakage system No. 3 was in the narrow part of the divide in the vicinity of, and upstream from, the powerhouse. The leakage outlets of system No. 3, leaks Nos. 5 to 19, inclusive, between May, 1927, and January, 1944, and the leakage after correction, are shown in Fig. 7. The inlets for this system were in the vertical face of the rock bluff upstream from the intake structures. Leak No. 5 which was the largest single leak, and the one farthest upstream from system No. 3, flowed strongly at all operating reservoir levels. The other leaks dried up progressively as the reservoir fell, the highest one ceasing first. Leaks Nos. 14 to 19 dried up when the reservoir fell below El. 778. This system of leakage contributed a flow of about 68 cu ft per sec at minimum reservoir and a flow of 286 cu ft per sec at full reservoir. (The white matter that stands out in Fig. 7(*d*) is asphalt and should not be mistaken for water leakage.)

Plan for Leakage Correction

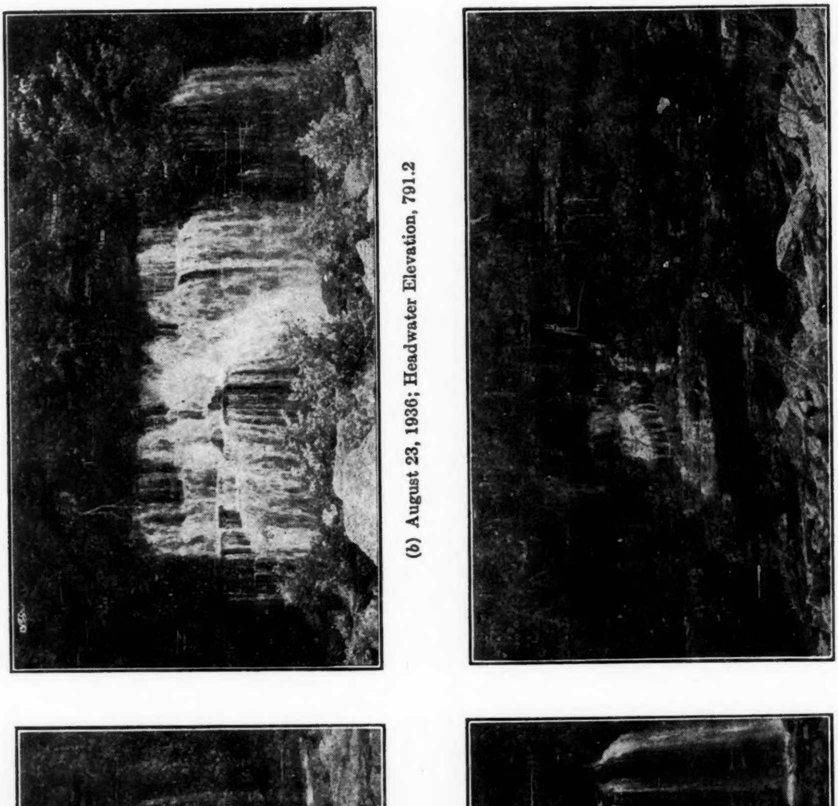
Experiments with asphalt and cement grouts, injected into holes drilled to intercept the leakage passages a short distance from the leakage outlets, were conducted by the TVA to test the feasibility of stopping this leakage without interfering with the normal operation of the reservoir. These experiments were successful in stopping most of leak No. 5, and the related data obtained indicated that the stopping of all the leakage was feasible economically.

The most natural concept of leakage correction on a problem of this magnitude would be to close up the inlets. However, at Great Falls, the inlets extended over a much longer traverse (approximately 10,000 ft) than did the outlets and the plugging of a known inlet would still permit other potential inlets to function. Since no physical harm can occur to the area between inlets and outlets, and since treatment near the outlets would in no way impair the hydroelectric structures, this apparently cheaper and more positive method was chosen.

The location of the grout holes had to satisfy two conditions: First, the holes had to be placed far enough from the leakage outlets so that effective grouting would not be interrupted by grout seepage from the leakage outlets too early in the grouting process; and, second, the holes had to be placed so that the vertical extent of the cutoff curtain would prevent leakage at all reservoir levels. The farther back from the outlets the holes were placed, the costlier would be the drilling, because the slope of the terrain rises sharply upward from the leakage outlets at the face of the bluff, whereas the level of the leakage channels as they approach the bluff remains about the same.

An average distance of 60 ft back from the bluff face was chosen for the experimental work. If the vertical extent of the cutoff were not involved, this distance would have been entirely adequate. Some few holes were as close as 35 ft to the leakage outlets, and they were asphalt grouted without serious loss of material before effective leakage correction. In general, a minimum distance of 50 ft was adequate for Great Falls.

The second consideration—the vertical extent of the cutoff—was controlled by the geologic formations at Great Falls. Where the shale dips below the full reservoir elevation, the vertical extent of the cutoff required was from the



(a) May 13, 1927; Headwater Elevation, 803.0

(b) August 23, 1936; Headwater Elevation, 791.2

(c) January 25, 1944; Headwater Elevation, 778.4

(d) March 26, 1946; Headwater Elevation, 799.4

FIG. 7.—PROGRESSIVE INCREASE OF LEAK NO. 5 BETWEEN 1927 AND 1944 AND AFTER CORRECTION IN 1946

contact of the two Fort Payne members to the base of the shale. Where the base of the shale is above reservoir level, the drill holes were grouted to El. 806.

For the first stage holes a 40-ft spacing was selected with succeeding second stage, third stage, and fourth stage holes on 20-ft, 10-ft, and 5-ft centers, respectively. The cutoff line was completed as follows: 10% at a final spacing of 20 ft, 60% at a final spacing of 10 ft, and 30% at a final spacing of 5 ft.

The effectiveness of asphalt grout as a remedy for stopping flowing water in underground channels was demonstrated during the experimental work at Great Falls. Water velocities to 6 ft per sec, and one leak having a discharge of 60 cu ft per sec, did not interfere with blocking the water-bearing cavities with asphalt. Where leakage existed at all reservoir elevations, grouting with asphalt was scheduled. Where leakage did not exist at low reservoir stages, grouting with Portland cement was considered possible.

The largest leakage outlets in any system were at the lowest part of the dip, decreasing in size as they developed up the dip, generally in a downstream direction. The location of leakage outlets along the dip was also influenced by the structural sags in the rock, and the development of individual outlets was always greatest at the low point of the sag. Leakage measurements, made after a substantial part of leak No. 5 was stopped, disclosed that no appreciable reduction in the total leakage at low reservoir levels was effected; but, at high reservoir stages, the total leakage reduction was approximately equivalent to the quantity of water contributed by leak No. 5. This condition probably existed because some of the channels of leaks Nos. 6 to 13 (which were part of the same system) were not utilized at the lower reservoir levels and the water normally discharged by leak No. 5 was simply diverted into these inactive channels.

At full reservoir, the channels were completely filled; and, to pass the water usually discharged by leak No. 5, new channels had to be formed—a slow process—or the velocities in the remaining outlets had to increase. There was some increase in the velocity of the discharge noted, and the enlargement of the channels was also stimulated, as evidenced by the appearance of muddy water as the leakage channels were progressively blocked. Accordingly, the large leaks at the lowest point of any system were sealed first and the water was forced up the dip into the smaller channels at higher elevations. The highest water velocity (as measured by computing the rates of travel to an outlet of fluorescein dye injected into the drill holes) never exceeded 6 ft per sec; the average was less than 2 ft per sec.

Holes were first asphalt grouted on 40-ft centers, and then on 20-ft centers, with an occasional 10-ft hole grouted to check the water flow. The large leaks were corrected with fewer holes than the smaller leaks because connections to the channels of the larger leaks were made more easily, and a positive injection of asphalt into the more developed channels was possible.

Cement-grouting operations were begun opposite the upstream end of leak No. 14 of system No. 3 and progressed downstream from that point. All facilities were concentrated in this area in an attempt to seal these leakage passages before normal reservoir operation required filling above El. 778, the elevation at which leaks Nos. 14 to 19 began to function.

The effectiveness of the cement-grouting program was difficult to ascertain, and only by grout acceptance and grout recovery in the second stage and the third stage holes was it possible to obtain any measure of the results. However, on November 22, 1945, heavy rains on the Great Falls watershed filled the reservoir, and the work accomplished to that time received its initial test. All leaks that were asphalt grouted remained closed. Good results were observed in the areas grouted with cement. Leakage correction opposite holes on 40-ft centers was 80% effective; and opposite holes on 20-ft centers the correction was 95% effective. When holes were spaced on 10-ft centers, the correction was practically 100% effective.

Fig. 8.—Typical Cores Drilled After Consolidation Grouting

Completion of the grouting program at low reservoir levels without interruption of the original work schedule was improbable during the flood season, and plans were changed to continue the work at existing reservoir levels using both asphalt and cement. Cement-grouting equipment and asphalt-grouting equipment were set up side by side so that a choice of material was available depending on the amount of leakage intercepted in the drill hole; but a completely cement-grouted hole on each side of an asphalt-grouted, or partly asphalt-grouted, hole was specified. Thus, cement grouts having water-cement ratios of 1.0 by volume or greater were pumped around the asphalt in an attempt to incase it, and to plug small openings into which asphalt probably would not penetrate. All remaining leakage was checked with this method.

Drilling Grout Holes

Diamond drilling was employed for sinking all holes. Casings consisting of 3.5-in. standard black pipe as long as 30 ft, and averaging 10 ft in length,

were required to reach the rock surface. In areas having numerous cavities and seams the drills were often blocked by sand, gravel, and blocky chert. These materials were usually most troublesome just above the contact of the upper and lower Fort Payne rock formation, and continued drilling caused the blocking of tools or the destruction of bits.

When this condition existed, drilling was stopped and the hole was pressure grouted with cement to consolidate the loose material. There was very little loss of cement through the leakage outlets, as most of the zone requiring this treatment had its inlets above reservoir level at the time the consolidation grouting was done. When drilling was resumed from 36 hours to 48 hours later, most holes were completed without further difficulty. However, if a second consolidation were necessary, it was done without hesitation. Typical cores showing the conglomerate texture obtained in reaming the cement grout from these holes are shown in Fig. 8.

The rate at which the diamond drill holes were completed determined the progress of the job. In all, six drills were available, but no more than five were in use at one time. The maximum number of drill shifts per day was fourteen. The drilling rates obtained are shown in Table 1. Diamond drilling costs were high because of the hardness of the rock existing at Great Falls. This hardness is explained by the large percentage of chert embedment in the Fort Payne formation. The average cost per foot of drilling for 32,536 lin ft was as follows:

TABLE 1.—Typical Drilling Rates

Item	Monthly Average		
	Minimum	Maximum	Average
Feet per shift hour	0.94	2.34	1.82
Feet per net drilling hour	1.33	3.01	2.27
Ratio, net hours to shift hours	0.71	0.83	0.80

Item	Dollars per linear foot
Drill Operation—	
Labor	0.94
Repairs (labor and material)	0.35
Drill tools	0.30
General plant and facilities	0.73
Casing 5,460 lin ft	0.20
Diamond drill bits	2.51
Gasoline (8,538 gal)	0.05
Miscellaneous	0.04
Moving and setting up (608 holes)	0.18
Total direct cost per linear foot	5.30

Asphalt Grouting

Asphalt was melted in five heaters of 500-gal capacity each, similar to those employed for highway work. The pumps were 4.5-in. by 3-in. by 4-in. air-driven, double-acting, reciprocating units equipped with iron ball valves, iron liners, and pistons. Oxidized petroleum asphalt was used for all grouting.

Deliveries were made in 30-ton to 40-ton carload lots scheduled for anticipated requirements so that dead storage was reduced to a minimum. Asphalt was packed in paper cartons, each containing about 1.5 cu ft of material.

Four asphalt-heating kettles were generally employed having a combined heating capacity of 80 cu ft per hr; three kettles were arranged in a bank, and from these the asphalt was pumped to the fourth kettle near the hole being grouted. Thus, by moving a single kettle approximately 80 ft each side of the three-kettle bank, holes 160 lin ft along the cutoff line could be grouted from one setup without danger of the asphalt freezing in the pipe lines. The asphalt-heating and asphalt-pumping arrangements are shown in Fig. 9. Holes were water tested and when no pressure was developed, or when connections to the leakage outlets were suspected, tests were made with fluorescein dye to determine the approximate velocity and the course of the water.

An important feature of the asphalt grouting as done at Great Falls was the position of the grout packer with respect to the water-bearing channels intercepted. The presence of other cavities in a hole above the main leakage channel, some active and some inactive, required the isolation of the lowest active cavity for successful grouting with the simplified equipment in use. The inclusion of several cavities below a packer resulted in the lowest cavities (usually the largest water-bearing channels) being unfilled with asphalt. This characteristic was observed many times at the beginning of the work by the emission of asphalt grout from the higher outlets, and in several places actual loss of asphalt to the surface occurred without sealing the water-bearing channels. Therefore, all packer settings for asphalt grouting were made as close to the water-bearing cavity as possible; and, where it was impossible to do so without crossing a higher cavity, pipe extensions were used below the packer to give the asphalt every opportunity of seeking the lowest water-bearing channel.

The rate of pumping varied considerably, and was largely a matter of judgment based on the behavior of the grout flow into the hole and its effect on the leakage through the connecting outlets. A pumping rate of 80 cu ft per hr was possible, but the average was usually between 40 cu ft per hr and 60 cu ft per hr. Rapid pumping was usually the rule where the water passages were small and the connecting outlets were scattered over a wide area. Where the water-bearing seams were large, the rate of pumping was usually slower. Pressures of 0.5 lb per ft of depth, measured from El. 810, were specified; but in the first stage and the second stage holes where most of the asphalt was placed pressure was of little consequence. When the connecting leakage was stopped, indicating that the "block" had been formed, pumping was discontinued regardless of registered pressure because cement grout in the same hole and in the next stage hole could be used to tighten the seal at the specified pressure.

The limited use of asphalt during any one continuous period of pumping was found to be advantageous in stopping the leaks (because it was soon evident that hot asphalt could travel distances beyond that necessary for effectiveness). By restricting the area covered by the asphalt and making repeated injections, the asphalt was permitted to build up the block within a

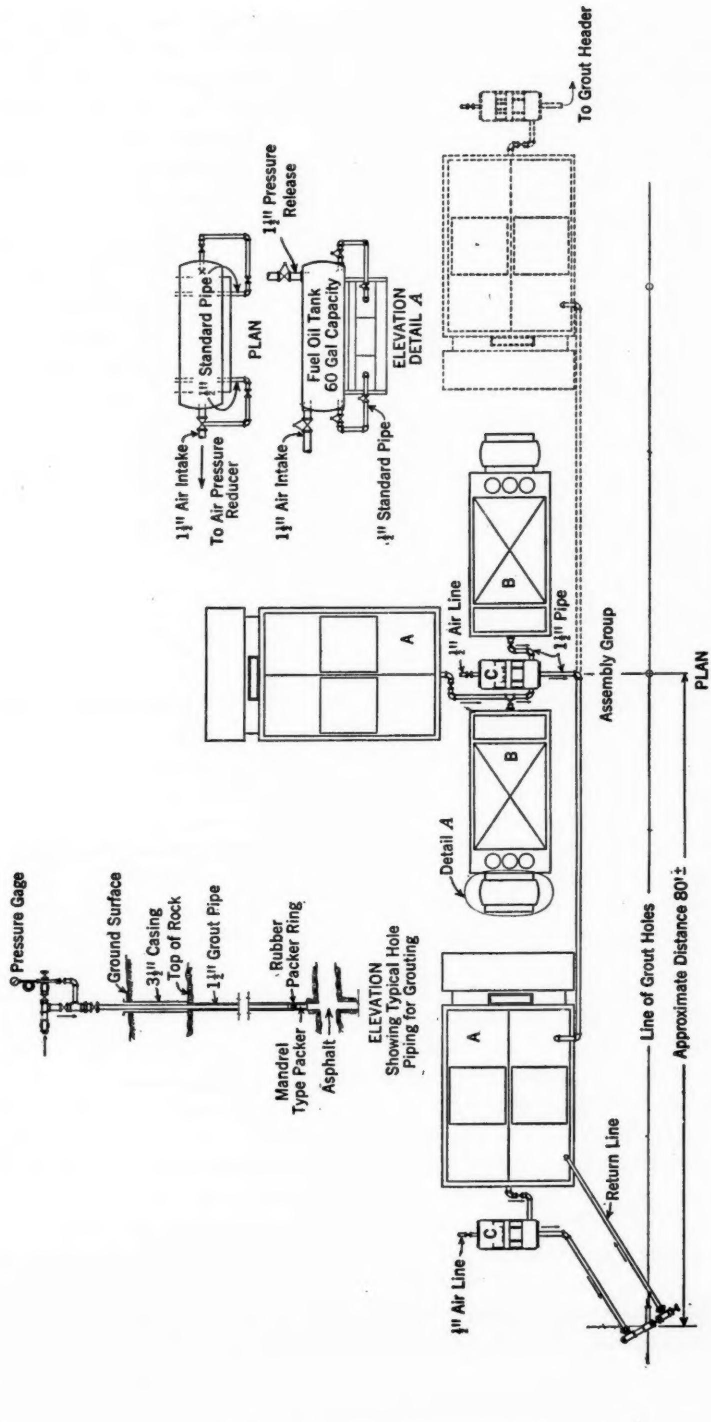


Fig. 9.—Asphalt-Grouting Equipment

reasonable distance of the hole. It also was found that injecting limited quantities of asphalt into a series of holes having the same outlet connections, before repeating the injection in any hole, gave rather spectacular results. For example, limited quantities of asphalt were pumped into a series of three holes connecting with a major part of the leakage at the right abutment (leak No. 20, Figs. 3 and 4), without noticeably changing the quantity of leakage; but, after a second application was injected into the same three holes, the entire leakage of 15 cu ft per sec was sealed in a period of about 12 hours. The results with this method were so rapid that it was continued, even though there was danger of losing a hole through interconnection. Some few holes were lost to pressure grouting in this way, but interconnection generally indicated that the cavities were sealed and that the desired results had been obtained. The quantity of asphalt placed at one continuous pumping was usually limited to from 600 cu ft to 1,000 cu ft. Greater quantities usually resulted in loss of asphalt through the leakage outlets—a condition which limited pumping tried to prevent. When asphalt did appear in the leakage outlets, pumping was stopped as nothing was gained from continued injection.

The temperature of asphalt varied between 300° F and 350° F. At temperatures lower than 300° F there was danger of asphalt freezing in the hole. Temperatures of 350° F were used where penetration of small cavities was desired, but in larger cavities a temperature of 300° F was ample. Asphalt was used only in the first three stages of grouting and only in cavities having flowing water.

The rate of usage largely controlled the unit cost of asphalt in place. A total of 548 crew shifts was used for asphalt grouting; and each crew contained a foreman, a pump operator, and six laborers. The average quantity of asphalt grout pumped by a single crew during a shift was 280 cu ft. The maximum placed by a single crew during a shift was 648 cu ft. Asphalt grouting was continuous over three 8-hour shifts daily whereas cement grouting was usually continuous over two consecutive 8-hour shifts daily. A total of 156,518 cu ft of asphalt was placed at Great Falls at a direct cost of $1.35 per cu ft, the distribution being:

Item	Dollars per cubic foot of asphalt
Asphalt, free on board Great Falls	0.77
Direct labor	0.25
Grouting equipment (depreciation and repairs)	0.06
General plant and facilities	0.19
Kerosene (87,400 gal)	0.05
Materials and miscellaneous	0.03
Total direct cost	1.35

Cement Grouting

Four cement-grouting units of the Boulder Dam (in Arizona and Nevada) type were installed. The Boulder Dam mixer is mounted on a steel frame

having two levels. On the upper level is a charging hopper and horizontal cylindrical mixing tank. The mixing mechanism consists of a horizontal shaft with paddles rotated by an air motor. The cement grout is discharged from the mixing tank through a 3-in. drawoff cock into a mechanically-agitated sump located on the lower level of the steel frame. The sump is a cylindrical tank 30 in. high and 36 in. in diameter. The agitating mechanism consists of a vertical shaft with horizontal blades set near the bottom of the tank, and the shaft is rotated by an air motor similar to that used for the mixing tank. Each grouting unit was equipped with two 7-in. by 5-in. by 10-in. air-driven, double-acting grout pumps.

Cement-grouting methods and procedures were standardized as much as possible during the course of the work. Constant observation of grout behavior as the grout was pumped into the drill holes, and as it appeared at the leakage outlets, permitted the establishment of rather rigid rules which achieved effectiveness with a restricted use of cement. The first packer setting, whenever practical, was made not more than 10 ft above the contact of the two Fort Payne members. The remainder of the hole was grouted in additional steps isolating the more open cavities with the step limit set at 20 ft.

Each packer setting was water tested to determine the resistance of the hole as an aid in the selection of the water-cement ratio. The pressure specified was the same as that used for asphalt grouting and this rule of pressure was maintained for most of the grouting. The maximum pressure used was 40 lb per sq in. with the exception of some third stage and fourth stage holes having sand-filled cavities which were grouted at pressures of from 50 lb per sq in. to 70 lb per sq in.

Every effort was made to bring the hole to its refusal pressure as quickly as possible in the first stage and the second stage holes. Third stage and fourth stage holes for the most part became check holes, and refusal of these holes under a gradual increase of pressure with a decrease in grout acceptance was sought. Continuous pumping without developing pressure on a hole was limited to from 300 bags of cement to 500 bags of cement, depending on the size of the cavity being grouted. The pumping of larger quantities of cement usually resulted in loss of cement through the leakage outlets. General limitations and general rules for grouting were as contained in Table 2. These rules were set up primarily to check the limits of cement usage, and variation within this range was permitted as each hole often presented its own difficulties that necessitated manipulation of pressures and mixes to secure refusal.

Calcium chloride was often used to accelerate the set of cement in an attempt to limit its travel. From 2% to 6% (by weight) of cement was the amount of calcium chloride added with percentages of from 4% to 6% being the most helpful. The set was accelerated with calcium chloride in an effort to secure a "block" somewhere in the cavity against which neat cement mixes could be pumped without loss. Calcium chloride was prepared in solution and added to the mixing water at the grout mixer.

Once pressure was obtained, every effort was made to close the hole by continued pumping to obtain the most effective distribution of the cement

under pressure. As soon as one half of the predetermined refusal pressure was indicated on the gage, the return line to the pump was connected and the flow of grout and pressure increase was controlled by stopcocks in the header pipe. Pressures were held for 20 min after refusal. After completion, holes requiring the use of heavy grouts (water-cement ratio by volume 0.8 or less) were rodded through the packer to make certain that the pipe and the hole remained open while grouting. Third stage and fourth stage holes and other tight holes were usually jetted to the previous completed packer setting and regrouted. Most of the holes did not accept cement on regrout and those that did usually accepted very small quantities. If rodding and jetting were not possible, because of obstructions, or the set of cement, and if there were reason to doubt the soundness of the grout (usually indicated by a rather abrupt refusal pressure), the hole was reamed and regrouted.

It was often necessary to inject the 300-bag to 500-bag limit of cement three or four times before filling the cavity in the first stage and the second stage holes sufficiently to develop pressure. One hole required seven injections before refusal was obtained. It was early discovered that without such restrictions vast quantities of cement could have been pumped without obtaining a closure.

As with asphalt, the unit cost of cement grouting was largely dependent on the rate of cement usage. A total of 866 crew shifts was required for the completion of the cement-grouting program; and each crew contained a foreman, a pump operator, and five or six cement handlers. The average quantity of cement grout pumped by a single crew during an 8-hour shift was

TABLE 2.—GUIDE TO THE

(Note: At Any Pressure Drop at Constant Rate of Pumping, Begin
at Constant Rate of Pumping,

Line No.	WATER TEST DATA		STARTING ONLY			FIRST CHANGE		
	Quantity[a] (1)	Grout pressure[b] (2)	Ratio,[c] w/c (3)	Bag limit[d] (4)	Grout pressure[b] (5)	Ratio,[c] w/c (6)	Bag limit[d] (7)	Grout pressure[b] (8)
1	Any	0	0.8	30 to 50	0	0.6	30 to 50	0
2	Any	0 to 50	0.8	50 to 100	50	0.6	50 to 100	50
3	Any	50 to 100	1.0	50 to 100	50	0.8	50 to 100	50
4	>10	100	1.0	100 to 200	100	0.8	100 to 200	100
5	>10	100	1.6	100 to 200	100	1.0	100 to 200	100

[a] Pumping rate of flow, in gallons per minute. [b] Percentage of the grout pressure specified. [c] Ratio of chloride ($CaCl_2$) mixes. [f] 5% $CaCl_2$.

288 bags. The maximum placed by a single crew during a shift was 866 bags. A total of 249,468 bags of cement was used in cement grouting the cutoff at Great Falls at a cost of $1.19 per bag. The distribution of this cost is as follows:

Item	Dollars per bag of cement
Cement, free on board Great Falls	0.65
Calcium chloride	0.01
Direct labor	0.22
Grouting equipment	0.07
General plant and facilities	0.18
Materials and miscellaneous	0.03
Reaming holes (4,837 ft at $1.45 per ft)	0.03
Total direct expense	1.19

Conclusions

The grouting at Great Falls succeeded in stopping about 98% of the reservoir leakage. Most of the remaining water loss, approximately 9 cu ft per sec, is in areas outside the limits of the cutoff and is so located that its correction is not economically justified, unless leakage materially increases in the ensuing years.

The leakage remaining within the limits of the cutoff-grouting line was only a matter of seepage 3 years after completion of work, and there was no indication that the grout curtain had deteriorated. As can be readily seen from Table 3, showing the total material placed, further quantities of grout could be forced into the rock either with grouts of higher water-cement ratio under higher pressures or through closer spaced holes. Neither of these procedures would have contributed to the actual amount of leakage corrected. Rather than continue with grouting that would not accomplish any positive results,

Use of Cement Grouts

with the Next Higher Water-Cement Ratio. At Any Pressure Rise Continue the Mix in Use.)

Second Change			Third Change			Remarks	Line No.
Ratio,[c] w/c (9)	Bag limit[d] (10)	Grout pressure[b] (11)	Ratio,[c] w/c (12)	Bag limit[d] (13)	Grout pressure[b] (14)	(15)	
0.6[e]	100 to 200	0	0.6[f]	100 to 200	0	Discontinue grouting	1
0.6[e]	100 to 200	50	0.6[f]	100 to 200	50	Continue grouting to refusal	2
0.6	100 to 200	50	0.6[e]	100 to 200	50	Continue grouting to refusal	3
0.6	100 to 200	50	0.6[e]	100 to 200	50	Continue grouting to refusal	4
0.8	100 to 200	100	0.6	100 to 200	100	Continue grouting to refusal	5

water to cement, by volume. [d] Bags (1 cu ft) of cement. [e] 4% calcium chloride mix, w/c = 0.6 for all calcium

the decision was made to postpone any further work until additional treatment is warranted.

The leakage corrected added approximately 10% to the output of the Great Falls plant. The annual value of this power is equivalent to a return

of more than 8% on the cost of the leakage correction. It is also estimated that, at the rate of annual leakage increase existing at the time of correction, an additional saving in power output of 1% per yr is being obtained.

TABLE 3.—ASPHALT AND CEMENT USAGE BY STAGES

Stage	Spacing[a] (ft)	Asphalt (cu ft)	Bags of Cement at Water-Cement Ratios (w/c) of					
			0.6[b]	0.6	0.8	1.0	1.6	2.0
First	40	115,291	25,628	31,504	26,204	18,274	2,526	116
Second	20	31,000	9,230	14,237	18,727	20,767	5,010	187
Third	10	10,227	3,163	6,963	10,909	23,423	10,275	3,332
Fourth	5	....	885	2,388	3,114	6,617	4,066	1,923
Total	....	156,518	38,906	55,092	58,954	69,081	21,877	5,558

[a] Distance between grout holes, in feet. [b] Calcium chloride ($CaCl_2$) mix.

The success of the leakage correction work at Great Falls was largely due to the topography of the Great Falls site. During the drilling of holes from 50 ft to 60 ft in rock, the effects obtained from grouting these holes were always visible from the bed of the Caney Fork River. Constant observation of the bluff containing the leakage outlets permitted visual inspection that made possible the establishment of rules and procedures for the limited usage of the grouting materials.

Acknowledgments

The leakage correction work was under the general direction of C. E. Blee, M. ASCE, chief engineer, TVA. The writer was construction engineer for the project and H. D. Towers was construction superintendent. Geologic investigations were under the direction of B. C. Moneymaker, Assoc. M. ASCE, chief geologist. TVA.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

LIGHT-WEIGHT PUMICE CONCRETE

Discussion

By G. B. Drummond, and J. P. Hawke

G. B. Drummond,[13] Assoc. M. ASCE.—It is fortunate that research and experience as described by Mr. Niederhoff have focused attention on pumice as a light-weight building aggregate. The days are passing when the small operator with a broken-down truck could haul a load of what he called pumice, untested and ungraded, and "palm it off" as a satisfactory material. In the present stage, however—the serious investigation of the properties of this material—there is the possibility that research is conducted to prove some preconceived ideas rather than to determine, disinterestedly, the basic data required for an honest design.

In New Mexico, where it is estimated that two fifths of the nation's source of pumice is found, there is pumice and also there is "pumice." Generally, pumice is found in extinct volcanic areas under an overburden of as much as 150 ft; but within these areas are encountered deposits of various chemical analysis. For example, the compositions in Table 6 come from three deposits in the same vicinity. A physical characteristic of pumice is that it requires special handling when being mixed. Its cellular structure, which causes it to float in fresh water (although its specific gravity is greater than unity), produces a tendency for the aggregate to separate from the mix, unless the water content of the aggregate prior to designing the mix has been carefully determined. Presaturation will fill the pores of the pumice aggregate sufficiently to cause all particles to approach the same specific gravity. Also,

TABLE 6.—Comparison of Pumice Chemical Characteristics

Element	Valencia	Cochiti	Jemez
Silica	77.90%	70.28%	71.67%
Alumina	11.28	13.54	12.64
Ferric oxide	0.86	2.06	2.30
Lime	0.80	1.96	0.60
Magnesium	0.36	1.18	0.70
Soda	3.64	4.14	8.00
Potash	4.38	3.33	Trace
Titanium	0.06	0.26	Trace
Loss on ignition	5.20	3.73	4.46
	104.28	100.48	100.37

Note.—This paper by A. E. Niederhoff was published in June, 1949, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1949, by Milton A. Karp.

[13] Civ. Engr., Albuquerque, N. Mex.

TABLE 7.—Design Data for Tentative Mixes of Pumice Concrete

Description	Compression Strength at 28 Days, in Pounds per Square Inch					
	3,000	2,500	2,000	1,500	1,000	500
Cement factor, in sacks per cubic yard	11.9	9.0	7.5	6.3	5.0	3.7
Ratio, w/c, for a 4-in. slump, in gallons per sack	4.6	5.9	7.5	9.5	11.4	17.0

better curing of the concrete will result when a portion of the water of hydration comes from within. Contractors in Albuquerque, N. Mex., have also learned that more uniform mixes have been obtained when the water and the cement are introduced into the mixer first, and the aggregate is added later.

TABLE 8.—Design Data for Pumice Concrete

Col. 1. f = specified compressive stress, in pounds per square inch;

Col. 2. γ_w = unit weight of wet pumice concrete mix, in pounds per cubic foot;

Col. 3. γ_d = unit weight of dry pumice concrete mix, in pounds per cubic foot;

Col. 4. f_c = working stress in compression, in pounds per square inch;

Col. 5. $\begin{cases} n = \text{modular ratio} = E_s/E_c; \\ E = \text{modulus of elasticity, in pounds per square inch;} \end{cases}$

Cols. 6. $K = \frac{f_c}{2} k\, j$;

Cols. 7. $k = \frac{1}{1 + f_s/n f_c}$;

Cols. 8. $j = 1 - \frac{k}{3}$;

Cols. 9. $p = \frac{f_c}{2 f_s} k$;

Cols. 6 to 9. $\begin{cases} f_s = \text{tensile unit stress in longitudinal reinforcement, in pounds per square inch; and} \\ a = \frac{f_s}{12{,}000} \times \text{(average value of } j\text{)} \\ \text{for use in } A_s = \frac{M}{a\, d}. \end{cases}$

f	γ_w	γ_d	f_c	n	(a) f_s = 16,000; a = 1.333 j				(b) f_s = 18,000; a = 1.500 j			
					K	k	j	p	K	k	j	p
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(6)	(7)	(8)	(9)
250	73.0	53.0	113	81.1	18	0.364	0.879	0.0013	....	....	....	....
500	74.5	54.5	225	54.1	42	0.432	0.856	0.0030	....	....	....	....
1,000	77.5	57.5	450	38.7	97	0.521	0.826	0.0073	93	0.492	0.836	0.0062
1,500	80.5	60.5	675	30.9	155	0.566	0.811	0.0119	149	0.537	0.821	0.0101
2,000	83.6	63.6	900	25.8	214	0.592	0.803	0.0167	206	0.563	0.812	0.0141
2,500	87.0	67.0	1,125	22.1	273	0.608	0.797	0.0214	263	0.580	0.807	0.0181
3,000	92.0	72.0	1,350	19.4	332	0.621	0.793	0.0262	321	0.593	0.802	0.0222

TABLE 8.—(*Continued*)

f	(c) f_s = 20,000; a = 1.667 j				(d) f_s = 22,000; a = 1.833 j				(e) f_s = 24,000; a = 2.000 j			
	K	k	j	p	K	k	j	p	K	k	j	p
(1)	(6)	(7)	(8)	(9)	(6)	(7)	(8)	(9)	(6)	(7)	(8)	(9)
1,000	88	0.465	0.845	0.0052	85	0.442	0.853	0.0045	81	0.421	0.860	0.0039
1,500	143	0.511	0.830	0.0086	138	0.487	0.838	0.0075	133	0.465	0.845	0.0065
2,000	198	0.537	0.821	0.0121	192	0.514	0.829	0.0105	185	0.492	0.836	0.0092
2,500	254	0.554	0.815	0.0156	246	0.531	0.823	0.0136	238	0.509	0.830	0.0119
3,000	310	0.567	0.811	0.0191	300	0.543	0.819	0.167	291	0.522	0.826	0.0147

Some difficulty has been encountered from those experienced only in working conventional concrete, because of the tendency of pumice concrete to "bleed." Troweling generally should be delayed until a slight set has been achieved.

Experience has shown that an all-pumice mix is limited to about 3,200 lb per sq in. because of the structural limitations of the aggregate. Consequently, a design basis of 2,000 lb per sq in. or 2,500 lb per sq in. is indicated. This may be obtained with approximately seven sacks of cement, with due regard to the water-cement ratio. Table 7 has been utilized in Albuquerque for the design of tentative mixes.

As noted by Mr. Niederhoff, ordinary tables for the design of reinforced concrete are not usable for the design of pumice concrete because the values of n are nearly twice those for ordinary aggregates. Table 8 for pumice concrete is based on investigations involving aggregates from the vicinity of Cochiti, N. Mex., but the data are applicable for other pumice materials of similar chemical content and proper gradation.

Acknowledgment.—In Table 7 the data were supplied by the Albuquerque Gravel Products Company and in Table 8, by the Pumice Aggregate Sales Corporation, both of Albuquerque.

J. P. Hawke,[14] Assoc. M. ASCE.—As with dense rock concrete, the best advantages of pumice concrete become available only with proper laboratory control of concrete production and placement. The laboratory control of pumice concrete has been materially aided by the formation of the Pumice Producers Association which was established to stabilize the quality and uniformity of the pumice aggregates produced by its members. In making preliminary laboratory design mixes particular consideration should be given to the practical factors of linear shrinkage and to the cement content required for proper workability.

Difficulties in placement due to the harshness of the mix when low cement contents are used have been cited by the author. The very successful use of air entrainment in improving the workability of lean and intermediate mixes of light-weight burned shale or expanded slag concrete has been reported.[15] This report would indicate that the workability of pumice concrete may be improved by air entrainment so that rich mixes are not required for maximum workability. The leaner mixes of pumice concrete would improve the economics of its use and also tend to reduce the linear shrinkage to be provided for in design.

The author has referred to several large buildings in the Los Angeles (Calif.) area in which light-weight pumice concrete was used. Light-weight concrete was also used in San Francisco, Calif., on the addition to the Matson Building, the Pacific Gas and Electric Company Annex Building, and the Standard Oil Company Annex Building. The use of light-weight concrete in these buildings has made possible very significant reductions in the structural dead loads. Since the lateral force requirements for earthquake resistance, in

[14] Berkeley, Calif.

[15] "Properties of Some Lightweight-Aggregate Concrete With and Without Air-Entraining Admixture," *Building Materials and Structures Report BMS 112*, National Bureau of Standards, U. S. Dept. of Commerce, Washington, D. C.

the local building code, are based on a stated fraction of the total column loads for each story, this reduction in dead load reduced the lateral forces to be provided for in the design. The reduced dead load was also important in connection with the foundation design. The engineers in charge of the foregoing projects selected haydite rather than pumice aggregates. This selection was made after extensive tests and studies and may have been influenced, to some extent, by the greater local experience in the use of haydite aggregate and by the readily available local source of supply.

The 1949 adjustments in the freight rate on pumice aggregates to San Francisco, together with improved technology in the use of pumice aggregates, may be expected to result in an increased interest in pumice concrete by engineers in this area.

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DISCUSSIONS

ANALYSIS OF PILE FOUNDATIONS WITH BATTER PILES

Discussion

BY GEORGE S. MURPHY, AND A. HRENNIKOFF

GEORGE S. MURPHY,[13] Assoc. M. ASCE.—A retaining wall supported wholly on vertical piles is subjected to translation and forward rotation about the toe. With batter piles present, however, the wall is subject to translation and backward rotation about the heel. This latter condition is brought about by the upward push of the heads of the batter piles, resulting from their rotation about an instantaneous center as the piles bend.

With the usual 30° batter, a forward translation of the pile head will produce an upward movement 0.577 times the horizontal movement. Since the height of the wall is usually about 2.2 times the base, the wall will tilt backward an amount equal to 2.2 times the upward movement of the toe. It follows that, with a forward translation, x (Fig. 14), the backward tilt of the top of the wall will be $2.2 \times 0.577\, x = 1.27\, x$, or about 25% more than the forward movement of the base.

Consider, again, the wall supported on vertical piles only. The active pressure behind the wall is well named, for, in order to produce any forward motion of the wall, the earth must follow it up. This it does by plastic flow acting over considerable periods of time.

Returning to a consideration of the wall with batter piles, a translation of the base of the wall must here, too, be followed up by the active flow of the earth mass, so that there is an approach to an amount of relative backward movement of the top of the wall equal to $5\, x/4$, rather than $x/4$.

The other extreme would be to assume that the plastic flow of the earth mass occurs only where the wall permits, so that only an active pressure is exerted on the wall except for the small section at the top where the absolute movement of the wall is backward, with a maximum of $x/4$ at the apex.

A third action is conceivable by assuming a cylindrical surface of shear failure from the heel of the wall to the surface of the fill, with the axis of rotation

NOTE.—This paper by A. Hrennikoff was published in February, 1949, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1949, by Jacob Karol; and September, 1949, by J. Owen Lake.

[13] Toledo, Ohio.

at the wall top (see Fig. 15). This would impose no more than active pressure on the vertical wall, but such an assumption has doubtful validity, especially since the axis of rotation also has a vertical translation.

It seems likely either that the true condition is intermediate between the first two, or that it is a mean of all three, with the several components having variable weights for different types of soil, but that, in any case, some passive earth pressure is developed by the wall when batter piles are used.

When the horizontal translation is sufficiently great, uplift is developed in the most forward vertical pile. In this case, even if the pressure, in excess of all the active horizontal pressure that can be developed, is neglected (as the author has done), some passive pressure must be developed on the upper surface of the base.

Taking into account the passive pressure developed against the back of the vertical wall, all the vertical piles may be hung from the base, as a result of which some greater-than-active pressure (Fig. 16) would develop along the entire top surface at the base. The only relief from this condition is that derived from the axial compression of the batter piles; but, in the case here considered, the depression so caused is less than the rise caused by the forward translation of the batter pile head.

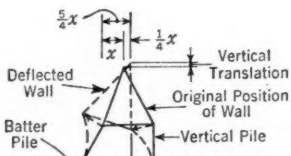

FIG. 14

FIG. 15

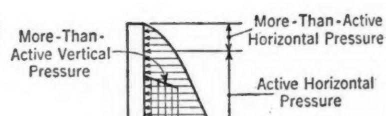

FIG. 16

In summary, it would appear that the author has not considered the development of horizontal and vertical passive earth pressures in the case of walls supported partly by batter piles, in so far as such pressures affect the distribution of pile loads and the stresses in the wall.

Such pressures are generated, at any rate, in the period during which the piles are developing deflections corresponding to the maximum earth pressures. Possibly, after the lapse of sufficient time, some of the passive resistance may leach away through the continued operation of plastic adjustments of the internal structure of the soil (such as by permanent consolidation), with the result that intensities equivalent to active pressures are approached at all points; nevertheless, it is reasonable to assume that at least some of the pressure intensities in excess of the active pressure will remain as elastic compression of the soil.

A. HRENNIKOFF,[14] Assoc. M. ASCE.—The misunderstanding that led to the criticism voiced by Mr. Karol is regrettable. The writer's sole reason for computing the approximate numerical values of the pile stiffness factors n, t_δ, t_α, and m_α, for different piles and soils, was to establish the range of the

[14] Prof., Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada

pile ratios for the purpose of demonstrating that the error involved in the writer's approximate theory (based on a single pile ratio r_1) is small when compared to the more exact theory, under virtually all possible pile-soil conditions (see under the heading, "Approximate Formulas for the Pile Constants"). Once the pile ratios are established there is no need to go back to the stiffness factors, since it is the ratios and not the stiffness factors which determine the distribution of loads among the piles of a given bent.

Accordingly, in his example, the writer makes use of the computed pile ratios, but not of the stiffness factors. In the paragraph immediately preceding Table 3 it is stated that the allowable axial pile load in all the cases listed in Table 3 is assumed to be the same—namely, 40 kips. From the viewpoint expounded by the writer, this assumption denotes that, in all cases, the piles are equally deformable axially—that is, they possess equal axial stiffness factors n. Their principal pile ratios $r_1 = \frac{t_\delta}{n}$, therefore, are directly proportional to the lateral stiffness factors t_δ; consequently, case 2*a*, which has a smaller r_1 than case 3*a*, involves a lower lateral stiffness and is more dangerous to the stability of the foundation than is case 3*a*, as is indicated by Table 3.

In comparing cases 2*a* and 3*a*, Mr. Karol assumes that they both involve the same lateral stiffness factors t_δ, and that a smaller value of r_1 in case 2*a* is caused by a greater axial stiffness factor n. By these assumptions he concludes correctly that case 2*a* is the safest; but, under the conditions prescribed by Mr. Karol, it would be incorrect to assign the same values of the allowable axial load to both cases, as is done and explicitly stated by the writer. Table 4 and its subsequent interpretation are affected by the same misunderstanding.

In connection with Mr. Lake's discussion, the writer wishes to point out that no progress, can be made, in the analysis of complex engineering problems, by considering all the factors in their entirety. The proper approach is to separate the individual factors and to account for them one by one. The writer's problem was limited solely to the analysis of pile behavior in a foundation under a known set of loads, and it did not extend to a consideration of different factors affecting these loads.

Mr. Lake is quite correct in stating that the variable passive pressure in front of the pile cap would contribute substantially to the safety of the piles. The writer admittedly did not consider this factor. Moreover, even if he had known how to allow for this variation of the passive pressure, he would have excluded it intentionally from his example in order not to obscure the picture of pile response to the known loads by an extraneous factor associated with variation of one of these loads. In an actual design, by all means, the passive soil resistance must be allowed for in the best manner known; but this has no bearing on the theory of pile analysis presented. Likewise, the writer's statement (borne out by the results of the example discussed) to the effect that the lateral pile resistance, even when very small, has a tremendous effect on the lateral stiffness of the bent, is not invalidated by the fact that a certain variable factor (the increased passive pressure on the cap) may also contribute in a substantial manner to the stiffness of the bent.

Incidentally, if the passive resistance is expressed as a known linear function of the horizontal cap displacement Δ_x, the solution of Eqs. 1 and the com-

putation of the pile loads and displacements are not complicated in any way, as compared with computations in the example presented. Even if the passive pressure on the cap were expressible as a known nonlinear function of Δ_x, Eqs. 1 could still be solved without much extra trouble by a few successive approximations.

Although the effect of the passive earth pressure on the resistance of the foundation to lateral loads is significant, it must not be overrated, or there would be no need for either the lateral pile resistance or the batter piles in pile foundations.

One must agree with Mr. Lake that there are cases in which it is not safe to rely even on a small lateral pile resistance; but one must not ignore the necessity of determining the lateral displacement of the bent. The writer's method is convenient for computing that displacement.

The paper by Donald Hamish Little,[11] and Mr. Lake's discussion of it,[12] deal with the design of "dolphin" piles, whose long projection into open water distinguishes them sharply from foundation piles. In the former the lateral deflection is an advantage and the bending stress is a matter of primary concern, whereas in the latter the deflection is a major fault, and the bending stress, as a rule, is of minor significance.

Mr. Lake made the following assumptions in connection with his method:

1. The pile is acted upon by the passive pressure in front and by the active pressure at the back, determined by the classical Rankine theory of pressure exerted by cohesionless materials on walls (observe that it is the walls and not the piles).
2. The side friction on the pile is allowed for by judgment in a manner not explained. No distinction seems to be made between round and square piles.
3. The earth pressure of full passive and active intensity is assumed to act for a limited depth along the pile, a length necessary to give the pile complete fixation, with no earth pressure extending farther down.

All these assumptions seem to be rather arbitrary, and, although it is possible that the effect of errors resulting from them may be not so great in application to the long free-standing "dolphin" piles, the adaptation of the method to the foundation piles embedded in earth does not appear likely to produce satisfactory results. Furthermore, the method would involve a greater number of constants describing each particular pile condition than the single constant used by the writer—the principal pile ratio, r_1.

In his discussion restricted to consideration of the example of the retaining wall analyzed by the writer, Mr. Murphy examines qualitatively the effects of the displacement of the base slab of the wall on the variation in values of the active and passive pressures exerted by the earth on the structure. Although in substantial agreement with Mr. Murphy's conclusions, the writer still feels justified in ignoring, in his example, the variations in the earth pressure for the reasons explained in connection with Mr. Lake's discussion.

[11] "Some Dolphin Designs," by Donald Hamish Little, *Journal*, Inst. of Civ. Engrs., November, 1946, p. 48.

[12] Discussion by J. Owen Lake of "Some Dolphin Designs," by Donald Hamish Little, in Supplement, *ibid.*, October, 1947, p. 453.

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DISCUSSIONS

FLOW AROUND BENDS IN AN OPEN FLUME

Discussion

BY L. O. CROES

L. O. CROES.[30]—The characteristics of flow around bends in an open flume have been made measurably clearer by the data presented in this paper. In this connection, mention should properly be made of the studies by H. Wittmann and P. Böss as reported at the first meeting of the International Association for Hydraulic Structures Research, held in Berlin, October, 1937.[31] Working in the Technical University of Karlsruhe, Germany, they based their computations on the potential theory, and produced results comparable to those obtained by Mr. Shukry.

In Europe, the investigation of flow around bends dates from 1868, when L. Fargue applied empirical laws to the regulation of the Garonne River in France. Subsequently, in 1902, Mr. Fargue developed the theory supporting his laws.

In Holland, where the rivers are occupied by an unusually intensive shipping industry, the subject of Mr. Shukry's paper is particularly applicable to the design of groins to guide the flow in both the upper and lower reaches. However, the problem is extensive in the lower reaches of the river, where the reverse action of salt tidewater disturbs the normal flow around bends in the river channel,[32] because normal spiral motion is possible only in the part of a cross section that is filled with water of the same specific gravity.

NOTE.—This paper by Ahmed Shukry was published in June, 1949, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1949, by Gilbert W. Outland, and Edward Silberman and Alvin G. Anderson.

[30] Civ. Engr., Rijkswaterstaat, The Hague, Holland.

[31] "Wasser und Geschiebebewegung in gekrümmten Fluszstrecken," by H. Wittmann and P. Böss, Julius Springer, Berlin, 1938.

[32] "Eenige Beschouwingen over De Waterbeweging in Beneden Rivieren," by J. J. Canter Cremers, *De Ingenieur*, September 17, 1921.

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DISCUSSIONS

APPLIED COLUMN THEORY

Discussion

BY HERBERT A. SAWYER, JR., AND E. C. HARTMANN

HERBERT A. SAWYER, JR.,[37] JUN. ASCE.—This well organized and stimulating paper can be mainly criticized for its quick rejection of the secant formula. The secant formula is a special case of the general column formula:

$$f = \frac{P}{A} + \frac{M\,c}{I_e} \qquad (62)$$

in which M is the actual maximum moment in the column from all possible causes. This expression is true even in the plastic range if the term c/I_e is properly modified. Contrary to what the author states in Section 7, the secant curve from this general formula has a very distinct relationship to the Euler column curve—in fact, the Euler-Engesser curve is only a special case of the secant curve. The general column formula also appears to agree very well with the experimental results[14] cited by the author in his treatment of eccentricity. Thus these results (and evidently the entire problem of eccentricity) seem to have, in the general column formula, a theoretical basis which should be recognized, and which serves as a direct tie between the two main parts of the paper—one on perfect columns, and the other on eccentric columns.

To verify these statements, the value of M in the general equation must first be found. For this discussion, it is sufficient to do this only for the cases of the paper—for any initial eccentricity, e, and any initial moment from transverse loads, M_b, both of which are symmetrically distributed with respect to the transverse center line of the column. Examples of these cases are shown in Figs. 22(a) and 22(b). Many investigators have evaluated M for various conditions of eccentricity, curvature, or transverse loading.[38,39] Prominent

NOTE.—This paper by F. R. Shanley was published in June, 1949, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1949, by William R. Osgood, and J. V. du Plessis; October, 1949, by Jacob Karol; and November, 1949, by E. P. Popov, and Jack R. Benjamin.

[37] Asst. Prof. of Civ. Eng., Univ. of Alabama, University, Ala.

[14] "Strength of Tubing Under Combined Axial and Transverse Loading," by L. B. Tuckerman, S. N. Petrenko, and C. D. Johnson, *Technical Note No. 307*, National Advisory Committee for Aeronautics, Washington, D. C., June, 1929.

[38] "Buckling of Elastic Structures," by H. M. Westergaard, *Transactions*, ASCE, Vol. LXXXV, 1922, p. 586.

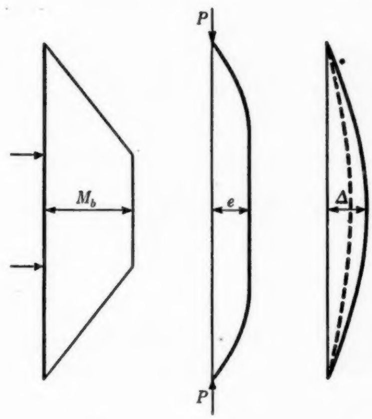
[39] "Numerical Procedure for Computing Deflections, Moments, and Buckling Loads," by N. M. Newmark, *ibid.*, Vol. 108, 1943, pp. 1161–1234.

among early investigators was J. M. Moncrieff,[40] M. ASCE, and the author[41] himself has suggested an approach to the problem.

The writer's reason for adding another basically similar column formula to the existing superabundance is that a formula was needed for this discussion which, using the author's notation, would apply generally to the author's various examples of eccentrically and transversely loaded columns. Existing formulas have been developed, for the most part, for specific types of transverse loadings or eccentricities.

Successive approximations will be used, although the principle of superposition is true only in the plastic range as applied subsequently, and for infinitesimally small values of M_b and e. The moment M will be found at the midheight of the column, where it is a maximum.

The primary moments at the midheight will be $M_b + P e$. These moments, distributed, for example, as shown in Figs. 22(*a*) and 22(*b*), each cause a transverse deflection, and, when added, would result in a deflection curve like Fig 22(*c*). The total midheight deflection from these moments, Δ, calculated by any standard method, would be



(*a*) (*b*) (*c*)

Fig. 22.—Examples of Initial Moments and Deflections

$$\Delta = C_b \frac{M_b L^2}{\pi^2 E_t I} + C_e \frac{P e L^2}{\pi^2 E_t I} \qquad (63)$$

in which C_b and C_e are constants depending on the shape of the curves corresponding to Figs. 22(*a*) and 22(*b*). Values of these constants, as defined by Eq. 63, are as follows:

Distribution of moment	C_b or C_e
Triangular (apex at midheight)	0.882
Third point trapezoidal (Fig. 22(*a*))	1.050
Quarter point trapezoidal	1.131
Rectangular (M_b or $P e$ constant)	1.26
Parabolic (vertex at midheight)	1.027
Sinusoidal (vertex at midheight)	1.000

The tangent modulus is accurate here only for infinitesimally small values of M_b and e, as shown by the author for perfect columns. For larger values, although not accurate, it is certainly more accurate than the elastic modulus.

[40] "The Practical Column Under Central or Eccentric Loads," by J. M. Moncrieff, *Transactions*, ASCE, Vol. XLV, June, 1901, p. 341.

[41] "Basic Structures," by F. R. Shanley, John Wiley & Sons, Inc., New York, N. Y., 1944, Chapter 19, p. 309.

The additional midheight moment, M_1, from the deflection Δ will be

$$P\,\Delta = M_1 = (C_b\,M_b + C_e\,P\,e)\,\frac{P\,L^2}{\pi^2\,E_t\,I} \qquad (64)$$

and the additional moment at any height will be P times the deflection at the height, producing a moment diagram geometrically similar to Fig. 22(*c*). This curve approximates a sine curve, with negligible error for the foregoing values of the constants, C_b and C_e, since its curvature usually approaches zero at either end and is maximum at the midheight. Assuming a sine curve, the additional midheight deflection from these moments will be $\frac{P\,\Delta\,L^2}{\pi^2\,E_t\,I}$, and the additional midheight moment from this deflection will be

$$M_2 = P\left(\frac{P\,\Delta\,L^2}{\pi^2\,E_t\,I}\right) = M_1\,\frac{P\,L^2}{\pi^2\,E_t\,I} \qquad (65)$$

This increment of moment (with sine curve distribution again assumed), in turn, causes another increment of midheight deflection. This deflection, multiplied by P, gives the next increment of moment:

$$M_3 = M_1\,\frac{P\,L^2}{\pi^2\,E_t\,I}\,\frac{P\,L^2}{\pi^2\,E_t\,I} \qquad (66)$$

This process may be continued indefinitely, each successive increment of moment containing an additional term $\frac{P\,L^2}{\pi^2\,E_t\,I}$. Totaling all these increments,

$$M = M_b + P\,e + (C_b\,M_b + C_e\,P\,e)\,\frac{P\,L^2}{\pi^2\,E_t\,I} \times\left[1 + \frac{P\,L^2}{\pi^2\,E_t\,I} + \left(\frac{P\,L^2}{\pi^2\,E_t\,I}\right)^2 \cdots\right] \qquad (67)$$

Summing up the geometric progression, and substituting this value of M in Eq. 62, the general column formula becomes

$$f = f_c + f_b + f_e + \frac{f_b\,C_b + f_e\,C_e}{\dfrac{\pi^2\,E_t\,\rho^2}{f_c\,L^2} - 1} \qquad (68)$$

in which $f_b = M_b\,c/I$; $f_e = P\,e\,c/I$; and $f_c = P/A$.

For the special case in which M_b and e are infinitesimal, when the term $\frac{\pi^2\,E_t\,\rho^2}{f_c\,L^2}$ becomes unity, or $P = \frac{\pi^2\,E_t\,I}{L^2}$, f will become infinite, indicating failure from instability. The curve of this failure will be the Euler-Engesser curve, the outermost curve of the infinite number of curves of f_c versus L/ρ, for the general formula, Eq. 68.

For the special case in which M_b and f_b are zero and e is constant throughout the length, C_e becomes 1.26. With this value the formula becomes practically

identical to the secant formula, the series of Eq. 67 becoming practically identical to a secant series.

Many arguments can be advanced as to whether this approach and formula are useful in showing failure above the elastic limit for a material without a sharp knee on its stress-strain diagram. As indicated previously, it certainly becomes less accurate in these regions, except for very small values of M_b and e. In any case, the value of this approach will be similar to the value of the fictitious modulus of rupture in predicting pure bending failure.

The best answer to this question is to compare the general formula with the test results. This procedure has been used for the series of interaction curves in Fig. 14. In doing this, only the values for f_{bo} (the modulus of rupture), the value $f_{co} = 60$ kips per sq in. for average ultimate stress for a short perfect column, and a derived value of E_t for $f_c = 50$ kips per sq in., as given elsewhere,[14] were used; all other points were calculated, using a form of the general column formula, giving "stress" at ultimate loading:

$$f_c + \left(1 - \frac{f_c}{f_{co}}\right) f_{bo} = f = f_c + f_b + \frac{1.050\, f_b}{\dfrac{\pi^2 E_t\, \rho^2}{f_c\, L^2} - 1} \quad \text{(69)}$$

The left-hand term of Eq. 69 (the "stress" at ultimate loading) is based on the reasoning that, if, for example, f_c is seven twelfths of f_{co}, failure will occur when the total bending stress exceeds five twelfths of f_{bo}. This result is probably not provable theoretically. Simplifying Eq. 69,

$$f_b = \frac{f_{bo}\left(1 - \dfrac{f_c}{60}\right)\left[1 - \dfrac{f_c}{\pi^2 E_t I}\left(\dfrac{L}{\rho}\right)^2\right]}{1 + 0.05\, \dfrac{f_c}{\pi^2 E_t I}\left(\dfrac{L}{\rho}\right)^2} \quad \text{(70)}$$

Curves plotted from Eq. 70 for $f_{bo} = 120$ kips per sq in., as indicated in Fig. 13 (and also the original[14] Fig. 10), did not check the experimental curves, especially at high eccentricity ratios. Further investigation showed that the experimental curves were plotted incorrectly. From the original source,[42] it can be deduced that the actual modulus of rupture varied from roughly 115 kips per sq in. to 135 kips per sq in., generally increasing with decreasing slenderness ratio and decreasing wall thickness. However, the original investigators[42] concluded that until some definite relationship was found it would be best to ignore this variation and use a constant value of 120 kips per sq in., thus making the experimental failure stresses relatively too high with respect to f_{bo}, and making the experimental curves plotted against this artificial limit err on the unsafe side for low slenderness ratios. Thus, through no fault of the author, the curves in Fig. 14 and Fig. 15 are inaccurate, on the unsafe side, in representing failure of compression tubing loaded transversely at the third points.

[42] "Strength of Tubing Under Combined Axial and Transverse Loading," by L. B. Tuckerman, S. N. Petrenko, and C. D. Johnson, *Technical Note No. 307*, National Advisory Committee for Aeronautics, Washington, D. C., June, 1929, pp. 6-7.

Curves were then plotted from Eq. 70 on the basis of a sliding value of f_{bo}, starting at 134 kips per sq in. for $L/\rho = 30$, and decreasing by 2 kips per sq in. for each increase of 10 in L/ρ, making $f_{bo} = 116$ kips per sq in. for $L/\rho = 120$. This very arbitrary variation seemed to fit the small amount of information[42] available concerning f_{bo}. It can be seen that the results, shown in Fig. 23,

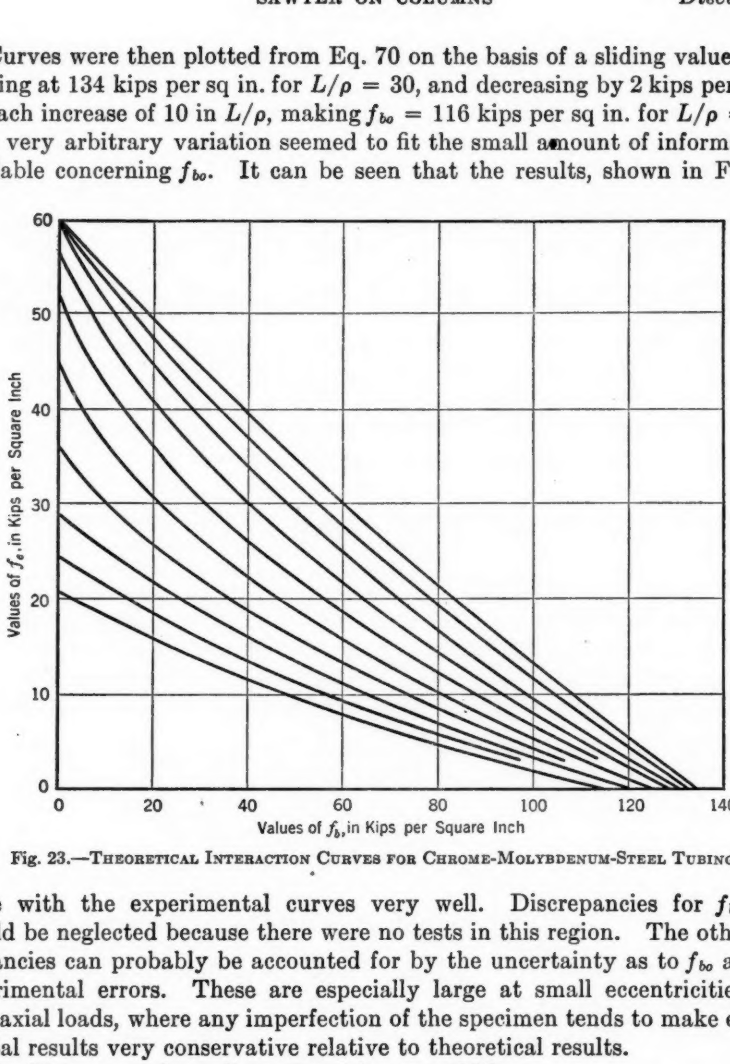


Fig. 23.—Theoretical Interaction Curves for Chrome-Molybdenum-Steel Tubing

agree with the experimental curves very well. Discrepancies for $f_b > 96$ should be neglected because there were no tests in this region. The other discrepancies can probably be accounted for by the uncertainty as to f_{bo} and by experimental errors. These are especially large at small eccentricities and high axial loads, where any imperfection of the specimen tends to make experimental results very conservative relative to theoretical results.

Eq. 70 in general nondimensional form would be

$$R_b = \frac{(1 - R_c)(1 - R^2{}_L)}{1 + (C - 1)R^2{}_L} \quad \text{(71)}$$

in which C is either C_b or C_s as defined previously;

$$R^2{}_L = \frac{f_c}{f_{cr}} = \frac{\left(\frac{L}{\rho}\right)^2}{\left(\frac{L}{\rho}\right)^2_{cr}} = \frac{\left(\frac{L}{\rho}\right)^2}{\frac{\pi^2 E_t}{f_c}} \quad \text{(72)}$$

$$R_c = \frac{f_c}{f_{co}} \quad \text{(73a)}$$

and

$$R_b = \frac{f_b}{f_{bo}} \quad \text{or} \quad \frac{f_e}{f_{bo}} \qquad (73b)$$

Eq. 71 is plotted for constant e, a truly eccentric column, in Fig. 24. The author's radial lines showing eccentricity are very useful for this type of chart. For Eq. 71 and Fig. 24 they must be plotted for

$$R_e = \frac{R_b}{R_c} = \frac{f_{co}}{f_{bo}} \frac{c}{\rho^2} e \qquad (74)$$

Similar diagrams could be plotted very quickly for any one of the values of the constants, C_b or C_e, by computing the ten or more points at $R_e = 0$ required to determine the straight lines for R_L.

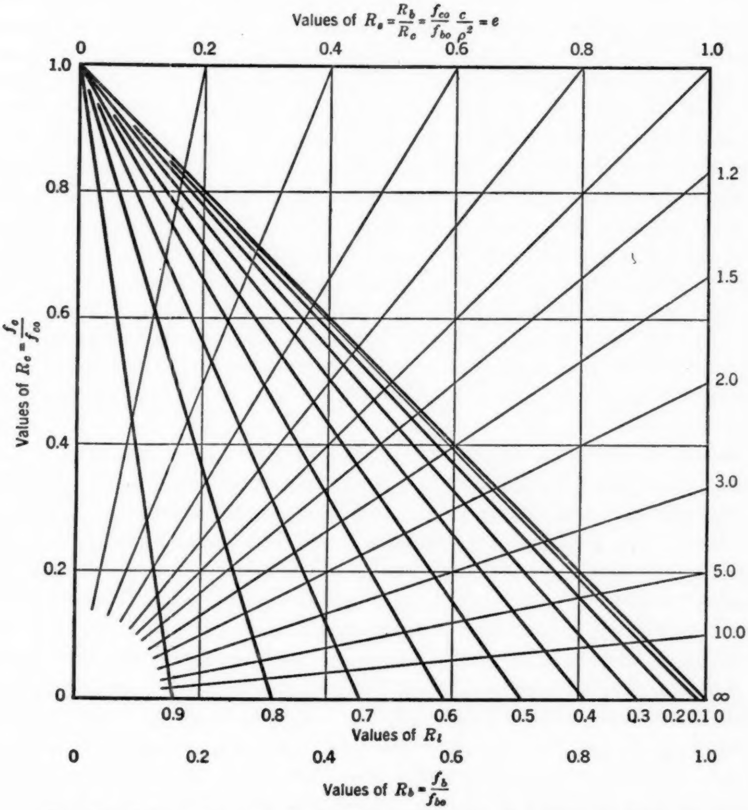


Fig. 24.—Reduced Interaction Lines for Eccentric Longitudinal Loading

Eqs. 71 and 72 support the author's prediction that, since R_L depends upon L/ρ and E_t, the lines of the nondimensional curve (for example, Fig. 24) would change with the value of the exponent n if drawn for L/ρ instead of R_L. Drawn for R_L, the lines are independent of n; and one set of nondimensional

lines (such as those in Fig. 24) are good for any column of constant $E I$ of any material with a certain type loading (such as the constant e of Fig. 24), within the limitations of the foregoing theory. These include, besides the limitations noted in the foregoing derivation, neglect of variation in tangent modulus (from size, shape, age, temperature, and direction and history of stress), failure in detail, and any imperfection not included in the terms M_b, C_b, e, and C_e.

To use Fig. 24 for a given material, f_{bo} or f_{co}, the stress-strain curve must be known. Then, opposite each value of R_c should be placed the square of the corresponding maximum slenderness ratio for the material, $(L/\rho)^2_{cr}$, computed from the Euler-Engesser formula (using the stress-strain diagram to obtain E_t). Then, for a given member and load, the allowable e may be found directly. If the member and e are given, R_c must be found by trial and error, two trials usually being sufficient. If the designer uses the concept of the factor of safety traditional to civil engineers, the design must be made on the basis of an ultimate load equal to the working load multiplied by the factor of safety, since f does not vary linearly with P.

Another check of Eq. 70 against experimental results was made, using the remaining data,[14] which was for duralumin tubing loaded in the same manner as the chrome-molybdenum-steel tubing. For these tests the value of the short perfect column strength had to be revised (for reasons similar to those for changing f_{bo} for the chrome-molybdenum-steel tests) to a value estimated at 38 kips per sq in. from the test data. The resulting theoretical and experimental curves did not check as well as the steel curves, the discrepancy being from -4 kips per sq in. to $+6$ kips per sq in. for values of f_b up to 55 kips per sq in. Even though this difference is well within experimental error,[43] this check of Eq. 70 against experiment is not very strong evidence of its usefulness for materials less elastic than chrome-molybdenum-steel. More evidence, pro or con, is needed.

A second criticism of the paper concerns the envelope equation of Section 5, in which it is stated that this equation could be used as a kind of over-all column curve for any material where weight saving is not too important. In Fig. 11 the envelope equation is plotted along with various standard column formulas, which form a "scatter band," and it is stated that the resemblance of the envelope curve to a parabola shows why parabolic curves have been found useful in representing the short column range.

Actually, the parabolic, straight-line, and Gordon-Rankine-Ritter formulas have applications entirely different from the envelope equation, also their applications are more practical. Any one of these formulas has been set up empirically for columns of a single material and class with a certain probable degree of accidental eccentricity (from warpage, rolling tolerances, accidental damage, and eccentricity from the tolerances of theoretically concentric end connections). On the other hand, the envelope equation applies only to perfect columns made of an infinite variety of materials. Certainly any

[43] "Strength of Tubing Under Combined Axial and Transverse Loading," by L. B. Tuckerman, S. N. Petrenko, and C. D. Johnson, *Technical Note No. 307*, National Advisory Committee for Aeronautics, Washington, D. C., June, 1929, p. 7.

agreement between the various empirical formulas and the envelope formula is entirely coincidental.

The coefficients of B in Eqs. 26, 27, and 28 actually vary widely from material to material and class to class, even in nondimensional form, to provide for different expectancies of imperfection. Also, the actual coefficients used for carbon steel structural columns, for example, show better agreement between the empirical formulas than Fig. 11 would indicate.[44]

The empirical formulas, easy to use, and taking into account unpredictable eccentricities for a given material and class of columns, will always have great practical value for the purposes for which they were developed. On the other hand, it is hard to see a design situation in which the envelope equation would be of value. For extraordinary materials and eccentricities, experimental results along with the general column equation (using the tangent modulus so ably advocated by the author, and maximum tolerable values of accidental eccentricity) should be used, the generalized Euler equation being a special case of this application.

The fundamental importance of accidental eccentricity to routine column design is illustrated by an example. A "perfect" 14 WF 202 column, 30 ft long, would take an ultimate load of more than 2,100 kips according to the generalized Euler formula. Taking into account chance eccentricities, the American Institute of Steel Construction (AISC)[45] gives a 3/8-in. "out-of-lineness" rolling tolerance for this column, and it would probably not be too conservative to add a constant e of 1/8 in. from all other causes. Using the general column formula with $f_{bo} = f_{co} = 40{,}000$ lb per sq in., the ultimate load is about 1,420 kips. The allowable load on this column, using the parabolic formula in the 1946 AISC specifications,[46] equals about 784 kips, which when multiplied by the usual factor of safety[47] of about 1.8 agrees roughly with the foregoing result of 1,420 kips.

E. C. Hartmann,[48] M. ASCE.—No structurally-minded engineer can read Mr. Shanley's excellent paper without finding much to stimulate his thinking, especially with regard to the significant part of the stress-strain curve for metals which lies above the range of purely elastic action. In his usual, clear, concise, and nonmathematical manner Mr. Shanley has covered some rather complicated concepts in a way which makes them more understandable, thus setting a good example to other engineering authors who do not always qualify for words of praise along these lines.

In the section of his paper dealing with the effects of eccentricity, Mr. Shanley has confined his attention to those members which fail in the plane of the applied bending moment. It might be well to point out that there is another classification of members, such as I-beams and H-beams, which may tend

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[44] "Resistance of Materials," by Fred B. Seely, John Wiley & Sons, Inc., New York, N. Y., 3d Ed., 1947, p. 229.

[45] "Steel Construction," AISC, New York, N. Y., 5th Ed., 1947, p. 285.

[46] *Ibid.*, 1946, p. 64.

[47] "Standard Specifications for Highway Bridges," Am. Assn. of State Highway Officials, Washington, D. C., 5th Ed., 1949, p. 242.

[48] Chf., Eng. Design Div., Aluminum Research Laboratories, Aluminum Co. of America, New Kensington, Pa.

to fail at right angles to the plane of the applied moment, thus complicating the problem beyond that which is covered in Mr. Shanley's treatment. If an I-beam, for example, is loaded to failure as a column with the end load applied in the plane of the web, but eccentric with regard to the centroid, it is highly probable (assuming no local buckling) that the I-beam will fail by buckling in a direction at right angles to the plane of the web.

The problem of the lateral buckling of eccentrically loaded I-shaped and H-shaped columns is not an academic one in civil engineering practice and deserves more attention than it has received in the past. Among those who have contributed are S. Timoshenko[49] in 1936; Bruce Johnston,[50] M. ASCE, in 1941; and H. N. Hill, Assoc. M. ASCE, and J. W. Clark,[51,52] Jun. ASCE, in 1948.

[49] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., 1936, p. 243.

[50] "Lateral Buckling of I-Section Column with Eccentric End Loads in Plane of the Web," by Bruce Johnston, *Transactions*, A.S.M.E., Vol. 63, 1941, p. A-176.

[51] "Alcoa Structural Handbook," Aluminum Co. of America, Pittsburgh, Pa., 1948, p. 57.

[52] "Lateral Buckling of Eccentrically Loaded I-Section Columns," by H. N. Hill and J. W. Clark (publication in *Proceedings*, ASCE, pending).

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DISCUSSIONS

END RESTRAINTS ON TRUSS MEMBERS

Discussion

BY JACK R. BENJAMIN, GEORGE WINTER, ABRAHAM SLAVIN, J. EDMUND FITZGERALD, AND JOSEPH S. NEWELL

JACK R. BENJAMIN,[19] Assoc. M. ASCE.—For their clear presentation of an exceedingly complex subject the authors are to be complimented. Although ultimate failure by instability is the criterion stipulated in the paper, applications to cases where unit stress controls, rather than instability, are important. The unit stress limit merely provides an alternate failure criterion.

The variation in stiffness and carry-over factor for columns under loading as contrasted to beams under loading is important in the design of building frames. In building frames the column loads are almost independent of girder moments so that a single adjustment in stiffness and carry-over factor is sufficient in an elastic analysis.

Two factors are neglected by the authors—influence of secondary stress moments and moments caused by initial curvature. Secondary stress influences will affect the buckling load of the entire truss. In addition, although the initial curvature of truss members does not influence stiffness or carry-over factors, it does add initial joint moments to the moment distribution procedure. These moments are approximately proportional to the axial loads and can be of the order of magnitude of 50% of the allowable moment in the member when initial curvatures specified by the current tolerances of the American Institute of Steel Construction are used. In an elastic analysis, these curvature moments can be significant. Whether or not these two influences reduce the buckling load of a truss a significant amount deserves investigation.

GEORGE WINTER,[20] M. ASCE.—Methods of investigating the buckling stability of structural frameworks are basically of two types: (a) "Classical methods" of solving the simultaneous transcendental equations which determine the buckling load; and (b) numerical methods of successive approximation. A very interesting historical review of the former was reported by Mr. Kav-

NOTE.—This paper by Harold E. Wessman and Thomas C. Kavanagh was published in September 1949, *Proceedings*.

[19] Asst. Prof. of Civ. Eng., Stanford Univ., Stanford University, Calif.

[20] Prof. and Head, Dept. of Structural Eng., Cornell Univ., Ithaca, N. Y

anagh[18] in 1948. For customary trusses with a considerable number of bars the classical methods are too cumbersome mathematically. The two numerical methods most promising for practical use are: (1) The Lundquist-Hoff[4,11,12] adaptation of the moment distribution approach, on which the present paper is based, and (2) the end restraint method developed by P. T. Hsu.[17] Method (2) has been published as part of an extensive study on buckling of trusses and rigid frames by the writer and his collaborators.[21] The contribution of the present paper consists in showing that the same end restraints which are found by method (2) can also be calculated from method (1). With regard to the implications of these various approaches the following comments can be made:

(1) In the Lundquist-Hoff method, ably restated by the authors, the stability of the truss is investigated at successively increasing loads until a load is reached for which the truss proves unstable. This procedure brackets the unknown exact critical load between the highest investigated load which proved stable and that load which showed instability. This bracketed range can be narrowed only by further time-consuming trials. The disadvantage of this approach is that each successive trial gives merely a new lower limit for the critical load so that the next trial load must be guessed, and may prove to be too high or too low by a considerable amount. (There are ways to reduce the required number of trials in the Lundquist-Hoff method; since no mention of them was made by the authors, they will not be discussed here.) In connection with this method the determination of restraints and effective lengths is wholly superfluous, as, in fact, is stated by the authors. After showing how to compute these quantities, the authors make no practical use of either of them.

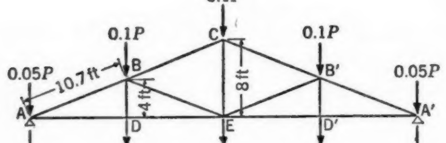


Fig. 7.—Illustrative Example

In contrast, the end restraint approach[17,21] gives a method of determining restraints and effective lengths directly without a detour through moment distribution. These quantities are then used to calculate critical loads directly. Here, too, the truss is investigated under successively increasing loads until the critical load is bracketed sufficiently closely. However, the advantage of this method is that each successive step gives both an upper limit and a lower limit for the critical load which reduces considerably the required number of trials. This fact is easily illustrated by the example of the

[18] "Instability of Plane Truss Frameworks," by T. C. Kavanagh, thesis presented to New York Univ., New York, N. Y., in April, 1948, in partial fulfilment of the requirements for the degree of Doctor of Engineering Science.

[4] "Stability of Structural Members Under Axial Load," by Eugene E. Lundquist, *Technical Note No. 617*, National Advisory Committee for Aeronautics, Washington, D. C., October, 1937.

[11] "Stable and Unstable Equilibrium of Plane Frameworks," by N. J. Hoff, *Journal of the Aeronautical Sciences*, January, 1941, p. 115.

[12] "Stress Analysis of Aircraft Frameworks," by N. J. Hoff, *Journal*, Royal Aeronautical Soc., July, 1941, p. 241.

[17] "Elastic Stability of Members in Trusses," by P. T. Hsu, thesis presented to Cornell Univ., Ithaca N. Y., in May, 1947, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

[21] "Buckling of Trusses and Rigid Frames," by George Winter, P. T. Hsu, B. Koo, and M. H. Loh *Bulletin No. 36*, Cornell Univ. Eng. Experiment Station, Ithaca, N. Y., April, 1948.

roof truss with heavy, suspended ceiling analyzed in detail by the writer and his collaborators,[22] as shown in Fig. 7.

For a trial load $P = 38$ kips, using end restraints, pertinent information was obtained for the three compression members, as listed in Table 4, in which k is the effective length coefficient; N_{38} is the compression force in a member caused by the loading $P = 38$ kips; N_{cr} is the critical load of the individual member for restraints determined for $P = 38$ kips; and $P_{N(cr)}$ is the loading P that would produce a bar force N_{cr} (for example, to have 177.9 kips in bar AB, a loading of $P = 40$ kips must be applied). Two facts are demonstrated by Table 4:

TABLE 4.—Roof Truss Analysis

Member	k	N_{38}	N_{cr}	$P_{N(cr)}$
AB.......	0.720	169.0	177.9	40.0
BC.......	0.885	112.2	117.8	39.8
BE.......	0.685	55.2	74.1	51.0

(a) The truss is stable at $P = 38$ kips since the critical bar forces are larger than those caused by this loading or, in other words, all $P_{N(cr)}$-values are larger than 38 kips. The latter value, therefore, is a lower limit for the exact critical load.

(b) On the other hand, the smallest of the $P_{N(cr)}$-values—that is, 39.8 kips—is an upper limit for the true critical load because, with increasing load, restraints decrease or (at best) remain practically constant so that, in turn, effective lengths increase, unless they also stay constant (see Fig. 6). Thus, with a load larger than 38 kips, say, 39 kips, all N_{cr}-values will be found smaller or, at best, equal to the tabulated values. Hence the corresponding $P_{N(cr)}$-values cannot exceed those found for 38 kips.

This particular trial, therefore, was found to bracket the exact critical load P between 38 kips and 39.8 kips, that is, within a 5% interval. For practical purposes, no further trial would be needed.

The reason for condition (b) is that with increasing axial load the effective rigidities of the compression members decrease. On the other hand, the rigidities of the tension members increase, but at a much slower rate and with generally negligible effect. Consequently, with increasing load on the truss the restraints provided by abutting members to the compression members generally decrease and result in a decreasing or, at best, a constant N_{cr}.

(2) Another substantial advantage of the restraint method, not mentioned by the authors, refers to the problem of design rather than that of analysis. To be sure, in a mathematical sense trusses fail as units by "one hoss shay" action. The authors state, correctly, that despite this action some "weaker" members of the truss contribute more to limiting its critical load than do other, more husky members. To illustrate: If, in the foregoing truss example, the critical load of slightly more than 38 kips was found insufficient for the given, actual design loads, the truss would have to be redesigned and strengthened. Theoretically the strengthening of any member will strengthen the entire truss. However, it is more effective to strengthen some members than others and the designer will want to change these members for maximum economy. The

[22] "Buckling of Trusses and Rigid Frames," by George Winter, P. T. Hsu, B. Koo, and M. H. Loh, *Bulletin No. 36*, Cornell Univ. Eng. Experiment Station, Ithaca, N. Y., April, 1948, pp. 35–40.

tabulated values show immediately that for this particular truss the members AB and BC should be strengthened, whereas substituting a heavier member for BE would have a small effect on the carrying capacity. This fact is evident because, for members AB and BC, the N_{cr}-values exceed the actual bar forces N_{38} only slightly, whereas the difference between these values exceeds 30% for member BE. The latter, therefore, has excess strength which is drawn upon by members AB and BC to supplement their own insufficient rigidity. The determination of such "weakest members" rather than the determination of the "weakest joints" discussed by the authors is important since it is not a joint but a member that must be redesigned for the desired effect. The determination of these "weakest members" is an automatic feature of the end restraint method and requires no additional computation.

(3) In some structures, particularly those subject to fatigue, the determination of actual maximum stresses at loads below the critical is important, and governs design. These stresses will exceed the value of P/A if lateral loads, eccentricities, or other imperfections induce bending. Such calculations are greatly facilitated by determination of end restraints for these subcritical loads. Corresponding methods (including tables and charts) were reported in 1948[21] as a further development of the end restraint method. A chart of the general type of Fig. 4[8] is also essential in connection with end restraint methods. However, negative as well as positive restraints can, and do, occur; and the chart developed at Cornell University, in Ithaca, N. Y., includes the range of such negative restraints, in contrast to Fig. 4.

(4) As stated by the authors, the methods in the paper apply only to frames without joint translation. The problem of effective length or of actual critical load, however, is of particular importance for rigid frames not restrained against sidesway. Fig. 8[21] shows the tremendous influence of sidesway restraint on the critical load of simple portal frames, for which

$$P_{cr} = \frac{\pi^2 E I}{(k l)^2} \qquad (24)$$

If such restraint is provided, effective length coefficients do not exceed 0.7 and 1.0 for the cases of fixed base or hinged base, respectively; without such restraint the corresponding maxima are 2.0 and ∞, respectively. The upper limit of $k = 2$ advanced by the authors in their last paragraph is seen to be correct, therefore, only for frames with completely fixed bases. It will be exceeded considerably for frames hinged or only partly restrained at the footings. The latter condition is true of most soils. Since buckling loads are inversely proportional to k^2, consideration of sidesway buckling is of paramount importance for rigid frames.

(5) In one of their concluding statements the authors seem to indicate that effective length determinations are superfluous for ordinary steel trusses and that compression members of such trusses can be designed on the basis of yield strength without regard to buckling. Some skepticism concerning this contention seems justified. As an example, at the critical load, values of P/A for

[8] "Elastically Encastred Struts," by N. J. Hoff, *Journal*, Royal Aeronautical Soc., September, 1936 p. 663.

the roof truss in Fig. 7—with slenderness ratios varying from 145 to 175 (that is, within the range allowed by current codes)—range from approximately 16 kips per sq in. to 25 kips per sq in. This range is considerably lower than the yield point of 33 kips per sq in. for mild steel, not to mention the still higher yield points that apply to the alloy steels mentioned by the authors. To be sure, current truss practice usually results in smaller slenderness values than those of this example; and, therefore, the design approaches more closely the situation depicted by the authors—probably due to the overly conservative column requirements in codes, particularly in older codes during whose lifetime

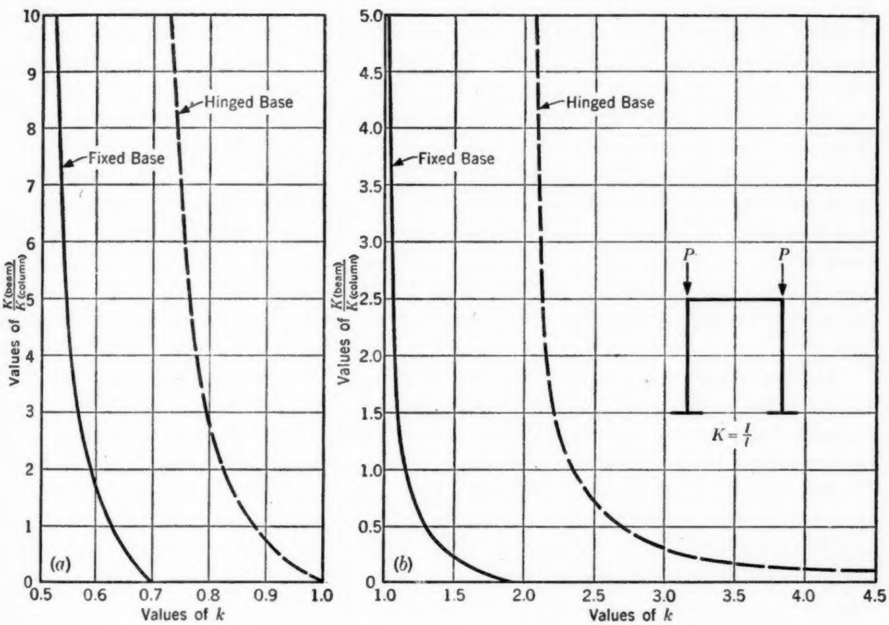


Fig. 8.—EFFECTIVE LENGTH COEFFICIENTS k FOR PORTAL FRAMES
(*a*) Portal Frame, Sidesway Prevented
(*b*) Portal Frame, Sidesway Permitted

present design habits have developed. It would seem that investigations of frame buckling such as those discussed are justified only if they promote the establishment of more realistic and, most likely, more liberal column design procedures. Once such procedures are introduced, a much larger number of structures will have proportions which require reasonably accurate determinations of effective length. Most design codes in the United States contain no reference to effective length variation. In contrast, the British code[23] dated 1948 makes explicit allowance for this factor, fifteen pages of that code being devoted exclusively to stipulations regarding effective lengths.

[23] "The Structural Use of Steel in Buildings," *British Standard Code of Practice, CP 113*, London, 1948.

ABRAHAM SLAVIN,[24] M. ASCE.—Average values of length reduction factors for compression members in civil engineering trusses are given by Friedrich Bleich,[25] M. ASCE, as $k = 0.80$. The Second Progress Report of the Special ASCE Committee on Steel Column Research[26] notes that from tests on pin-ended columns the suggested value of k is 0.78. E. H. Salmon[27] mentions that the k-value for practical end conditions probably lies between 0.56 and 1.00 and suggests the value of 0.78. For light roof trusses having members attached by small gusset plates to adjacent members of less stiffness, he proposes the value of $k = 1.0$. The late Leon S. Moisseiff,[28] M. ASCE, gave, as the nearest simple equivalent to an actual compression member in civil engineering structures, a column with partly restrained ends for which $k = 0.75$, or the average of the theoretical value 0.50 for fixed ends and the theoretical value 1.0 for pinned ends in an ideal column. A. S. Niles, Assoc. M. ASCE, advises that for airplane trusses of welded jointed tubular sections, designed by ultimate load formulas, the value of $k = 0.70$ in the prime compression member (highest value of L/j at critical load) was practically substantiated by static tests to failure made at an army airfield in United States during 1920 and later. This value is practically equal to that theoretically derived for a compression member having one end restrained and the other end pinned, although both ends of the truss column were actually subject to an intermediate degree of restraint. J. S. Newell reports that, for most of the airplane truss frames he analyzed, the critical load corresponded with the prime compression member having a k-value of about 0.70.

The foregoing results are only a few of those from numerous reports regarding restraint coefficients. Perhaps the difficulty, if any, is that many of the published column formulas do not indicate the type of end restraint, and it is assumed that the column formulas are for the practical compression members which may have varying degrees of end restraint. In fact, the column formula specified by the American Institute of Steel Construction does not indicate the type of end restraints; and, from discussion with well-known structural research engineers, the general assumption is made that the formula applies to a practical column with no definite or theoretical value for the end restraints.

In the fourth paragraph of their paper, the authors state:

> "It is not generally recognized that, as loads are increased on a framework, the end restraints acting on a compression member in the framework do not remain constant."

From the experience of the writer he believes that these facts are fairly widely recognized. Computed restraint factors from the stability studies of the plane

[24] Cons. Civ. Engr. and Architect, New York, N. Y.; and with Dept. of Civ. Eng., Polytechnic Inst. of Brooklyn, Brooklyn, N. Y.

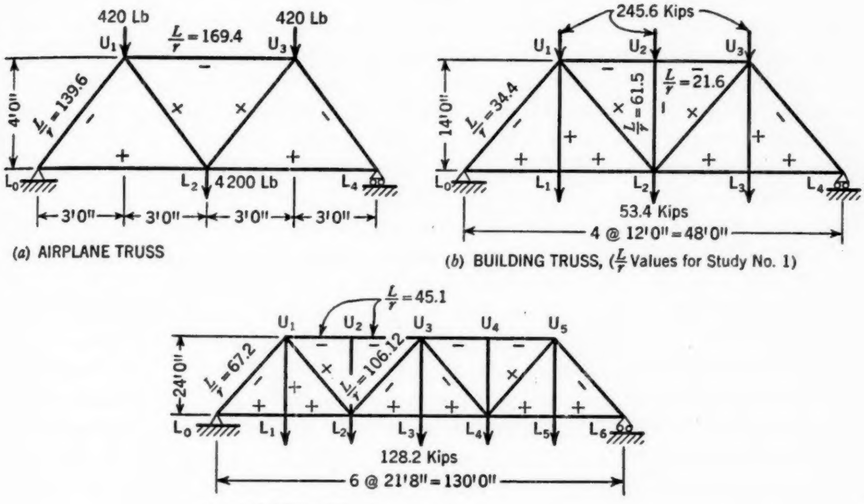
[25] "Die Knickfestigkeit Elasticher Stabverbindungen," by Friedrich Bleich, *Der Eisenbau*, April, 1919, p. 83.

[26] "Steel Column Research: Second Progress Report of the Special Committee," *Transactions*, ASCE, Vol. 95, 1931, p. 1201.

[27] "Materials and Structures," by E. H. Salmon, Longmans, Green & Co., London and New York, Vol. 2, 1937.

[28] "Design Specifications for Bridges and Structures of Aluminum Alloy 27 S-T," by Leon S. Moisseiff, Aluminum Co. of America, Pittsburgh, Pa., Revised Ed., March, 1940.

TABLE 5.—CRITICAL VALUES FOR TRUSSES [a]



(*a*) AIRPLANE TRUSS

(*b*) BUILDING TRUSS, ($\frac{L}{r}$ Values for Study No. 1)

(*c*) HIGHWAY TRUSS

Study	Design formula	Yield point[b]	$\bar{E}$	Critical load factor	Critical Values for Member: L_0U_1 Stress[b]	L_0U_1 k	U_1U_2[c] Stress[b]	U_1U_2[c] k
				(*a*) Airplane Truss				
1	Ultimate	36,000	E_u	1.0310	24,307	0.715	19,320[d]	0.704[d]
2	Ultimate	36,000	E	1.0452	24,648	0.758	19,589[d]	0.701[d]
				(*b*) Building Truss				
1	Old AREA[e]	36,000	E_u	2.5553	35,035	0.834	35,681[d]	0.765[d]
2	Old AREA[e]	40,000	E_u	2.8372	38,900	0.829	39,617[d]	0.781[d]
3	AISC[f]	36,000	E_u	2.1509	34,969[d]	0.766[d]	34,294	1.532
4	AISC[f]	40,000	E_u	2.3868	38,805[d]	0.768[d]	38,055	1.524
5	Old AREA[e]	36,000	E_T	2.57791	35.343	1.530	35,998[d]	0.572[d]
6	AISC[f]	36,000	E_T	2.21109	35,948[d]	0.715[d]	35,254	2.185
7	AISC[f]	33,000	E_T	2.0272	32,960[d]	0.704[d]	32,321	2.228
8	AISC[f]	36,000	E_u	2.1344	34,701[d]	0.860[d]	35,072	1.064
9	AISC[f]	40,000	E_u	2.3680	38,499[d]	0.861[d]	38,911	1.073
				(*c*) Highway Truss				
1	AREA (1943)[e]	36,000	E_u	2.3983	32,209	0.847	34,536[d]	0.786[d]
2	AREA (1943)[e]	33,000	E_T	2.2865	30,708	1.150	32,926[d]	0.703[d]

[a] All Warren type trusses except studies 8 and 9, Table 5(*b*), which are Howe type trusses. [b] Pounds per square inch. [c] In the airplane truss, Table 5(*a*), this becomes member U_1U_3. [d] Values for the member with the highest ratio L/j. [e] Old specifications of the American Railway Engineering Association, as distinguished from the 1943 specifications. [f] American Institute of Steel Construction.

trusses of three types[14] are given in Table 5. The k-values were obtained by the simple method of applying the Euler formula with the reduced modulus.

The airplane truss in Table 5, designed by ultimate load formulas, is the same as that used by the authors (except for engineering notation) and by Messrs. Niles and Newell.[29] The building truss by E. J. Squire, M. ASCE, and the highway truss,[14] both designed by working load formulas, are included as technical supporting data for values of k, and as an illustration of the relation of critical stresses to yield point values. For the Howe type of building truss (studies 8 and 9, Table 5(b)), the interior web members are reversed in direction to parallel the end posts. In this discussion, E_u refers to the effective modulus evaluated from the basic column formula; E_T is the tangent modulus evaluated from a typical stress-strain curve for structural steel; E_D is the double modulus; and E is the elastic modulus. In the case of the airplane truss, E may be used instead of E_u with an increase in value of the critical load of only 1.5%.

For both studies of the airplane truss, k may be taken as practically equal to 0.7. The first four studies of the building truss, and the first study of the highway truss, on the basis of the conservative modulus E_u, indicate a k-value close to 0.78, which has been suggested instead of unity for pin-ended columns in civil engineering structures to allow for the friction at the pin joints. For building trusses of the Howe type (studies 8 and 9), on the basis of E_u, $k = 0.86$. For the studies using E_T, with the exception of study 5, the k-values are about 0.7, or the same as those evaluated for the airplane truss with the modulus concept of E_u. The restraint coefficients will vary with loading, but the value at the critical load is of interest. All the data in Table 5 were evaluated and compiled prior to, and independently of, the preparation of the paper by the authors and were reported to them as part of the group research work.

In presenting Eq. 2, the authors mention that the tangent modulus is now generally accepted by aeronautical engineers and that it has also been accepted by the Column Research Council of the Engineering Foundation. However, in the computations for the airplane truss, the authors (in applying Eq. 15), as well as the writer, used the reduced modulus E_u. Since the effective modulus value is important for computing the critical loads and the corresponding restraint coefficients for the compression members in the framework, comment regarding the evaluation of the reduced modulus is warranted. The value of E_u is lower than the value of the tangent modulus and is intended to replace the Engesser and the von Kármán formulas that make no allowance for the effect of the practical column as compared to that of the corresponding ideal column. With a low value of the reduced modulus, the numerical value of L/j is increased and the computed theoretical critical load is a conservative value. This method of computing the reduced modulus is given by Messrs. Niles and Newell.[30] The report of the Army-Navy-Civil Committee on Aircraft Design Criteria[31]

[14] "Stability Studies of Structural Frames," by A. Slavin, thesis presented to New York Univ., New York, N. Y., in May, 1948, in partial fulfilment of the requirements for the degree of Doctor of Engineering Science.

[29] "Airplane Structures," by A. S. Niles and J. S. Newell, John Wiley & Sons, Inc., New York, N. Y., 3d Ed., Vol. II, 1943, p. 306.

[30] *Ibid.*, Vol. I, 1943, pp. 332–334.

[31] "Strength of Aircraft Elements," *ANC-5*, Army-Navy-Civil Committee on Aircraft Design Criteria, Revised Ed., Washington, D. C., December, 1942 (as amended August, 1946).

also states that the modified Euler formula does not have much practical importance in determining the short column curve, but that it is of practical interest in connection with the determination of the effective modulus that can be used to compute instability stresses. N. J. Hoff[32] mentions that the specification of a short column formula establishes a uniquely determined connection between the reduced modulus and the elastic modulus. W. Prager,[16] Eugene E. Lundquist and Claude M. Fligg,[33] and Mr. Moisseiff and Frederick Lienhard,[34] M. ASCE, also have used E_u.

Recommendations for the tangent modulus by F. R. Shanley[3] and O. H. Basquin[35] are based on the ideal unit columns. The recommendations by Mr. Shanley are essentially a verification of the results of prior laboratory work on ideal columns by the Aluminum Research Laboratories[36,37] at New Kensington, Pa. K. Borkmann[38] suggests an expedient estimate of the effective modulus on the basis of a linear relation between the proportional limit and the yield point. The writer[14] proposes an effective modulus based on a linear relation between seven tenths of the yield point and the yield point, which is intended for structural steels. If seven tenths of the yield point value is above the known value of the proportional limit, it does not apply and in its place the Borkmann modulus is suggested. It is the considered judgment of the writer that allowance should be made in the modulus value for the difference between the practical column and the ideal column.

In the derivation of their formulas, the authors use a spring constant s, and M is the applied moment equal to S, the stiffness for the far end fixed. Since a unit moment is applied for each loading state to determine the unbalanced moment or stability factor r, the assumption that $M = S$ is a limited condition. It is evident that with increased loading the stiffness of the members will decrease, although the same unit moment is applied in the computation procedure. The writer's computations[14] indicate that, for the airplane, building, and highway trusses (Table 5), with the far ends fixed (as is commonly assumed in the analysis of the entire framework), ΣS is positive for the loading above the critical determined by the stability $r = 1.0$. For the trusses in Table 5, the difference in critical load on the basis of the stiffness criterion $\Sigma S = 0$ and the series criterion $r = 1.0$ is about 17% for the airplane truss, about 0.4% for the building truss, and about 1.4% for the highway truss.

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[32] "A Note on Inelastic Buckling," by N. J. Hoff, *Journal of the Aeronautical Sciences*, April, 1944, p. 164.

[16] "The Buckling of an Elastically Encastred Strut," by W. Prager, *Journal*, Royal Aeronautical Soc., November, 1936, p. 833.

[33] "A Theory for Primary Failure of Straight Centrally Loaded Columns," by Eugene E. Lundquist and Claude N. Fligg, *Technical Report No. 582*, National Advisory Committee for Aeronautics, Washington, D. C., 1937.

[34] "Theory of Elastic Stability Applied to Structural Design," by Leon S. Moisseiff and Frederick Lienhard, *Transactions*, ASCE, Vol. 106, 1931, pp. 1052–1091.

[3] "Inelastic Column Theory," by F. R. Shanley, *Journal of the Aeronautical Sciences*, May, 1947, p. 261.

[35] "Tangent Modulus and the Strength of Steel Columns in Tests," by O. H. Basquin, *Journal of Research*, National Bureau of Standards, September, 1924, p. 381.

[36] "Column Strength of Various Aluminum Alloys," by R. L. Templin, R. G. Sturm, E. C. Hartmann, and M. Holt, *Technical Paper No. 1*, Aluminum Co. of America, Pittsburgh, Pa., 1938.

[37] "Typical Tensile and Compressive Stress-Strain Curves for Aluminum Alloy 24 S-T, Alclad 24 S-T, 24 S-RT, and Alclad 24 S-RT Products," by R. L. Templin, E. C. Hartmann, and D. A. Paul, *Technical Paper No. 6*, Aluminum Co. of America, Pittsburgh, Pa., 1942.

[38] "Charts for Checking the Stability of Compression Members in Trusses," by K. Borkmann, *Technical Memorandum No. 800*, National Advisory Committee for Aeronautics, Washington, D. C., July, 1936.

For the building truss in Table 5(*b*), the relative stiffness of the joints under increase in loading is shown in Fig. 9. Joint U_2 has the highest relative stiffness at the design load up to a load factor of about 2.52, beyond which, with load increase, it becomes the joint of least stiffness. Thus, the weakest joint is not the one having the least stiffness at the design load but rather the joint having the greatest rate of loss of stiffness with loading increase. The authors have also verified this statement with reference to the airplane truss. Since the joint stiffness is the sum of the stiffness of all members entering the truss

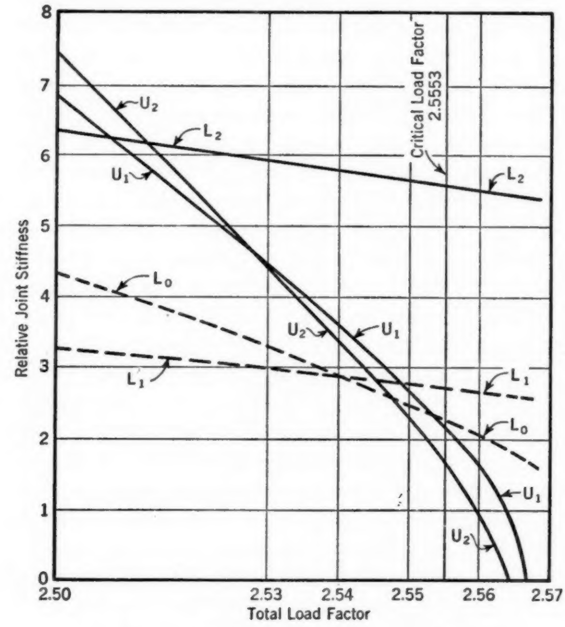

Fig. 9.—Joint Stiffness; Building Truss, Study 1

joint, the data in Fig. 9 which are related to the truss in Table 5(*b*), indicate that the stocky compression members at joint U_2 ,which have high unit stress, lose their contributing stiffness at the greatest rate because of the buckling tendency under increased loading; and, in turn, their respective restraint values are also reduced. The total stiffness of each joint has been found to decrease in relation to the prevalence of compression members composing the joint, and not in proportion to the increase in loading.

In the paragraph presenting Eq. 2, the authors claim that the buckling load in civil engineering trusses for a fixed-end column may be only slightly greater than that for the same column with pinned ends. The data in Table 6 from studies 1 of the trusses in Tables 5(*b*) and 5(*c*) cast some light on this question. The member having the highest ratio of L/j at the critical load, on the basis of E_u, is considered.

In stability computations, $L/j = \pi$ for pinned-end columns, and $L/j = 2\,\pi$ for theoretical fixed-end columns. A narrow range in load factors between these two conditions of end restraint is indicated for the cited typical civil engineering trusses. However, as noted by Mr. Hoff,[12] in actual frameworks the end fixity is elastic and is not perfectly rigid, and the upper limit of $L/j < 2\,\pi$ for a compression member with both ends rigidly fixed cannot be reached.

TABLE 6.—Comparison of Fixed–Ended Columns and Pin–Ended Columns for the Primary Compression Truss Members Having the Highest Ratios L/j at Critical Loads

(Yield Point 36 Kips per Sq In.)

(*a*) Member U_1U_2, Building Truss					(*b*) Member U_1U_2, Highway Truss		
Load factor	Stress (lb per sq in.)	Ratio, L/j	Percentage range[a]	Load factor	Stress (lb per sq in.)	Ratio, L/j	Percentage range[a]
2.539	35,455	3.142	0.0	2.335	33,625	3.142	0.0
2.555[b]	35,680	4.108	+0.63	2.398[b]	34,536	3.977	+2.70
2.568	35,865	6.288	+1.14	2.460	35,425	6.372	+5.35

[a] Percentage range in load factor. [b] Critical load factor.

The first and last values of the data for each truss in Table 6 may be used to approximate the buckling load, from which k-values may be estimated, since there is a small range in load factors between the upper and lower values of L/j. The data in Table 6 also indicate that the critical compressive unit stresses are close to the yield point. This reasoning does not apply to the airplane truss in Table 5(*a*), in the same sense that it applies to the airplane truss in Fig. 5, because the compression members have high slenderness ratios and they fail at unit stresses considerably below the yield point.

In Fig. 6, the plot of r versus the load factor for the airplane truss applies to a unit moment at joint U_1. With application of a unit moment at joint L_0, the r-values are lower up to the equivalent critical load value at $r = 1.0$ and then are higher. For verification of the equivalence of the critical load, where $r = 1.0$, the unit moment should be applied separately to each joint at which compression members enter. It will be observed that the critical load, where $r = 1.0$, is the same regardless of the joint at which the unit moment is applied, as discussed by the authors in theorem 3. It is evident, however, that, for a truss frame in which all members are interdependent, at the critical load the frame fails as a unit regardless of the reserve strength in any members, since the continuity of the frame is destroyed.

The authors credit Mr. Lundquist[4] for the stability criteria and Mr. Hoff[11,12] for his contribution. The analysis method should be called the Lundquist-Hoff method. Although the work by Mr. Lundquist[4] is shown as

applicable to member groups of a truss and although it may be applied to the entire truss frame, the procedure by Mr. Hoff is definitely applicable as a valid solution for the entire framework. In addition, credit is due to Messrs. Niles and Newell[29] for their moment distribution method which modifies the usual procedure. In the joint at which the unit moment is applied, the carry-over values are held against further distribution, which thus materially shortens the computations for the stability factor *r*.

The authors note that, for compression members in steel trusses, with the usual range of slenderness ratios, the buckling load practically coincides with the yield point of the steel regardless of the end restraints; and therefore any elaborate analysis of stability and end restraint is unwarranted for a steel truss. It is well known that steel (whether carbon steel, silicon steel, or nickel steel) has a well-defined yield point and that the elastic modulus E is the same for all regardless of the different values for proportional limit and yield point. However, the statement by the authors should be qualified to apply to civil engineering structures designed on the basis of working load formulas. It does not apply to steel trusses used in airplanes, which are designed by ultimate load formulas and have slender compression members that fail at stresses considerably below the yield point.

The data in Tables 5 and 6 indicate definitely that, for civil engineering trusses of structural steel, the critical compressive unit stresses are practically at the yield point. Therefore it is evident that, for these trusses, the necessary stability analysis for the critical load is futile. This does not apply to the airplane truss (as in Table 5(*a*)), because the critical compressive unit stresses are considerably below the proportional limit. From a plot of column curves for structural carbon steel, for both pinned and fixed restraints,[14,18] it is readily observed that (in the usual range of L/r-ratios for civil engineering sections) the critical unit stress is close to the yield point. The relation between the buckling unit stress of a unit structural steel column (of the usual stocky section in civil engineering structures) and the yield point of the material has been discussed for many years. The suggestion has been made that the buckling unit stress be approximated at nine tenths of the yield point for columns in civil engineering structures. It is also known that a column, as part of a framework, is subject to conditions different from those in the member acting as a unit.

Prior investigators, particularly in the field of aeronautic structures, have indicated that, when a steel frame is being loaded, its compression members will drain possible reserve strength from the adjacent members. This fact is mentioned by the authors and has been independently verified by the writer.[14] Since previous research reveals that, for a unit column of the civil engineering type, the buckling unit stress is close to the yield point, by intuition it follows that the buckling unit stress of a column in a framework must be practically at the yield point because of the stress contributed by the possible reserve strength in the adjacent members. In fact, in discussions with a number of structural analysts the same general opinion was expressed to the writer although it was

not based on stability calculations. However, this relation does not apply to the airplane truss example, used by the authors and the writer, and it should be noted that stability analysis is required for such structures. The critical load factor for the airplane truss cited is 1.0309 in the paper and 1.0310 according to the writer—indicating a 3% safety factor on the basis of design load. It is evident therefore that a stability analysis is required for the airplane truss.

The authors state that it is advisable to use small simplified member groups of the truss instead of the entire framework to determine the critical load, and that the illustrative example is from an arrangement proposed by the writer.[14] The grouping for the airplane truss is shown in Fig. 10. Because of the limited size of the truss all the members are employed when the terminals are twice removed from the joint where the unit moment is applied. For simplicity in calculation fixed ends are used throughout. The grouping in Fig. 10 was recommended to Mr. Kavanagh because it was found applicable to both aeronautical and civil engineering trusses. The calculations for r are simple, and satisfactory values for k result in from close to the true values of the critical load. Based on the generalized application of the Saint Venant theory, it is evident that, the farther the terminals are from the joint where the unit moment is applied, the closer the results will come to the true answer. Furthermore, the use of a grouping with terminals twice removed from both ends of a compression member requires considerable calculations with different r-values at each end, up to the value of the equivalence r. In this case, there will be approximately the same variation from the true critical load as for the grouping in Fig. 10. With terminals twice removed, contribution to the stability limit is allowed from the twice removed tension members and from any possible low-stressed compression members. For the grouping in Fig. 10, the operation is applicable only to the members that enter the joint where the unit moment is applied, because no moment is carried back from the twice removed terminals to the joints once removed. Denoting the stiffness ratio of the members as S_r, instead of as $K/\Sigma K$; and, with C as the carry-over factor, the unbalanced moment r for a unit moment at joint U_1 in Fig. 10 is equal to

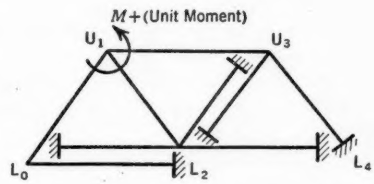

Fig. 10.—Member Group, Airplane Truss

$$r = \Sigma \begin{bmatrix} (S_r\, \overline{U_1L_0})\ (S_r\, \overline{L_0U_1})\ C^2 \\ (S_r\, \overline{U_1U_3})\ (S_r\, \overline{U_3U_1})\ C^2 \\ (S_r\, \overline{U_1L_2})\ (S_r\, \overline{L_2U_1})\ C^2 \end{bmatrix} \qquad (25)$$

Thus, a simple calculation instead of a lengthy moment distribution is available for satisfactory values of r.

The developments by the authors, and this discussion, are both based on planar trusses, with buckling within the plane, assuming the effect of joint translation to be negligible.

J. EDMUND FITZGERALD,[39] JUN. ASCE.—An original contribution to, and a survey of, the various methods of determining truss capacities are presented in this paper. It is to be regretted, however, that the authors did not include an example of an actual steel truss, preferably indeterminant, in order that the reader might be enabled to judge their method as applied to commonly encountered design problems.

The paper derives its value from the authors' presentation of a method of handling trusses with rigid joints whose $\frac{L}{i}$-values are greater than those normally encountered in practice, since for the usual truss the authors state:

> "* * * investigation has demonstrated clearly that the buckling loads for steel building and bridge trusses * * * are so close to * * * the yield stress * * * that there is absolutely no need for buckling load analyses."

The writer believes that the worth of the paper lies in the mathematical confirmation of the theory of limit design as originally presented by J. A. Van den Broek,[40] M. ASCE.

As will be shown, the writer disagrees with the limitation set by the authors and believes that there is no need of a buckling analysis for trusses with a slenderness ratio greater than those commonly encountered in practice. As a case in question, consider the truss shown in the example in Fig. 5. This truss is unquestionably in the slender range since the values of L/i for its members vary between $L/i = 140$ and $L/i = 170$.

In spite of the mathematically correct statement by the authors that "* * * when a truss buckles all members tend to fail simultaneously" (which would be difficult indeed to demonstrate in the field or in a laboratory), the writer will employ dictum (3), from Mr. Van den Broek's paper to solve the truss in Fig. 5.

In dictum (3) it is stated:

> "When in a * * * n-fold redundant structure, n redundants are stressed to * * * their critical buckling load, the deformations involved are of the order of magnitude of elastic deformations until an $(n + 1)$th member has reached * * * critical buckling capacity."

In order to provide for the redundants that are produced by the moment restraints at the various joints, the analyst may use the common design practice of assuming the effective lengths of the members as $L' = 0.75\ L$—that is, $K = 0.75$. For members with $\frac{L}{i}$-ratios between 100 and 200, this assumption is borne out quite well. For $\frac{L}{i}$-ratios less than 100, using the yield stress will give excellent results. This is in agreement with the authors' recomendations. For L/i greater than 200, the structure will probably be poorly designed. Thus, the assumption of 0 redundants is incorporated in the effective length and the designer may proceed to the analysis by limit design.

Since the truss shown has 0 redundants, $n + 1 = 1$, and the problem resolves itself into finding the weakest member—that is, in the authors' terms,

[39] Asst. Prof. of Civ. Eng., North Dakota State College, Fargo, N. Dak.

[40] "Theory of Limit Design," by J. A. Van den Broek, *Transactions*, ASCE, Vol. 105, 1940, p. 638.

finding the member which approaches failure the fastest (thus decreasing the reserve of strength in the structure to zero) and produces incipient failure.

Considering the truss as loaded with a load P at joint E and with loads 0.1 P at joints B and C the individual bar stresses are determined by statics. The entire calculation is shown in Table 7.

The values for the ultimate or buckling unit load were taken by interpolation from data published by Mr. Van den Broek. For $\frac{L}{i}$-values of 100 or so, the $\frac{L'}{i}$-ratio will give an ultimate unit load approximately equal to the yield point value.[41] The areas of the members and other pertinent data can be taken from the original Van den Broek paper.

The authors' overload factor of 1.03 represents a 1% difference from the result in Table 7. Actually, the variation of the physical and geometric constants of the truss itself is likely to exceed any error introduced by choosing a L'-value that is in error. The authors have found that the correct L'-value at failure is about 0.702. This represents a difference of almost 7%.

TABLE 7.—ANALYSIS OF THE TRUSS IN FIG. 5

Member	Stress	$\frac{L}{i}$	$\frac{L'}{i}$	Ultimate strength	Load	P	Overload factor
AB,CD	$-0.750\ P$	140	105	24,000	3,200	4.270	1.017 governs
BE,EC	$+0.625\ P$	140	....	36,000	4,800	7,740	1.840
BC	$-0.875\ P$	170	127	19,000	3,520	5,620	1.340
AE, ED	$+0.450\ P$	167	....	36,000	4,800	10,670	2.540

The tentative conclusion to be drawn from all this is that the theory of limit design may be used for the analysis and design of rigid trusses of any reasonable degree of slenderness ratios with the ensuing results in close agreement with those obtained by the more laborious, but mathematically more rigorous, method of this paper. From the designer's point of view, the paper should serve as another confirmation of the validity and universality of the theory of limit design. It should be mentioned that the authors' use of the ultimate load for design purposes rather than the allowable stress concept shows again how rapidly this newer and infinitely more logical procedure is taking hold. Let it be hoped that, in time, the building codes will recognize it at least as an alternate method of design.

The statement (second paragraph beyond Eq. 2)—

> "* * * the abutting members, instead of creating an end moment on the compression member as it starts to buckle, may have moments presented to them by the buckling member"

—seems to state the same thing twice—action and reaction, etc. Actually, the statement could be clarified to explain that the abutting members, instead of impressing a moment on the compression members of one sense before buckling

[41] "Theory of Limit Design," by J. A. Van den Broek, John Wiley & Sons, Inc., New York, N. Y., 1948, p. 85, Fig. 51.

occurs, which tends to buckle the column, will impress a moment of opposite sense on the column after it buckles, which tends to reduce the degree of buckling. This is the same phenomenon which occurs as reversed eccentricity when testing a flat-ended eccentric column.

In the "Synopsis," the authors make a statement which seems to persist in any discussion about columns. It represents a confusion of physical effect that even the Column Research Council has been guilty of on past occasions. To state that "* * * end restraints decrease and that the effective lengths increase as loads are increased * * *" is simply to state the same physical effect in two ways. The stiffness of the column may be determined by using a fixed value of end restraint, say, $S = 0$, and considering the effective length, L', to vary; or, the effective length may be assumed constant, say, $L' = L$, and the degree of end restraint may be considered to vary. The stiffness effect in the writer's example was provided for by assuming $S = 0$ and $K = 0.75$. This was done to use the results of tests on pinned-end columns in the selection of buckling unit loads. It certainly represents a needless complication to consider both the variation in K and S since they are directly related;[42] thus:

$$K = L' = \frac{2}{3}\left(\frac{S + 3\,I/L}{S + 2\,I/L}\right) \qquad (26)$$

The writer should like to congratulate the authors on their clarity of presentation, and hopes that in the closing discussion there will be an example of the type mentioned at the beginning of this discussion.

JOSEPH S. NEWELL.[43]—The authors have summarized, clearly and concisely, the state of the art of stability determination as it is currently applied to trusses. Their excellent paper leads the reader to the conclusion that stability analyses of frames or trusses involve computations which, although not difficult, are tedious. It certainly indicates that further research is desirable to develop simpler methods of evaluating stiffness coefficients for various members or various joints. Table 2, which is used in determining such coefficients for trial loads 1.0119 times those used in the basic example,[13] is by no means short, yet it omits thirteen of the nineteen cycles actually made. Even so, it yields little definite information beyond the fact that the criterion shows the structure under investigation to be stable at this load and, hence, that the computations must be repeated for at least one increase in load if the limit of stability is to be determined exactly.

It is no reflection on the merit of the paper that so much work produces so little actual information. It is, rather, an indication of the present state of the art, an indication that great rewards may be had from studies that will enable analysts to approximate critical loads or stiffness coefficients by less tedious methods.

[42] "Interrelation of Certain Structural Concepts," by Camillo Weiss, *Transactions*, ASCE, Vol. 111, 1946, p. 391.

[43] Prof. of Aeronautical Structural Eng., Massachusetts Inst. of Technology, Cambridge, Mass.

[13] "Airplane Structures," by A. S. Niles and J. S. Newell, John Wiley & Sons, Inc., New York, N. Y., 3d Ed., Vol. II, 1943.

Perhaps the conclusion—that for compression members in steel trusses with the usual range of slenderness ratios the buckling stress coincides practically with the yield point of the steel regardless of end restraints—will eliminate the need for stability studies for many bridge and building structures. It will not do so in aircraft design, however, nor in any structure that includes members subject to local buckling at stresses below the yield point of the material. These constitute an important fraction of the structures subject to stability investigations.

What should be regarded as the yield point? J. A. Van den Broek,[44] M. ASCE, has shown test results which indicate the peak of the curve before the "drop-of-the-beam" value to be the desired yield. Many engineers may accept this value, or the drop-of-the-beam value for mild steels, but what is to be used for alloy steels or aluminum alloys whose yield stresses are established arbitrarily?

Theorems 1, 2, and 3 cannot be stated with sufficient emphasis. They present points which many engineers have never considered, and few have analyzed to the extent that they clearly understand the action of a truss when it reaches a condition of instability. Whether the "series" or the "stiffness" criterion be used, it is important to realize that in a truss which has become unstable the reserve strength and stiffness in every member and joint have been exhausted, the unstable members undergo deflections, and the unstable joints undergo rotations, which are not proportional to the loads on the structure. The authors are to be congratulated on the clarity of their discussion of these points.

Now that they have summarized the long and tedious procedures in current use, it is hoped that the authors, or others inspired by their work, can evolve approximate methods which, although they may not eliminate a final "series-criterion" analysis, will reduce the preliminary or trial-and-error computations through which the critical load is estimated. The current procedure is not difficult to understand, but it is far too tedious to be practicable for routine application to structures having a large number of members.

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[44] "Euler's Classic Paper 'On the Strength of Columns,'" by J. A. Van den Broek, *American Journal of Physics*, July–August, 1947, p. 309.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

AERODYNAMIC THEORY OF BRIDGE OSCILLATIONS

Discussion

By David J. Peery, Alexander Klemin, and Abraham Slavin

David J. Peery,[25] M. ASCE.—The writer had the privilege of discussing and checking much of the author's work during the preparation of the paper, which was completed in essentially the present form early in 1947. The author has combined extensive research into the fields of aerodynamics and vibrations with his knowledge of suspension bridges. This paper presents a reliable and practical analysis. The following discussion will further confirm the validity of using tests on static models.

An oscillating suspension bridge is under the action of elastic, inertia, damping, and aerodynamic forces. The elastic and inertia forces may be computed accurately, and the natural modes of vibration resulting from these forces have been obtained by several investigators and verified by model tests. The structural damping forces are more difficult to determine, since models do not simulate the true conditions in the prototype. Fortunately it is conservative to assume a limiting case of no damping.

The aerodynamic forces on a thin airfoil oscillating in an ideal fluid have been obtained theoretically. Theodor von Kármán, Hon. M. ASCE, and W. R. Sears[26] have shown that these forces may be resolved into three components as follows:

(*a*) Forces that would be produced if the wake had no effect, called "quasi-steady" forces;

(*b*) Forces produced by the reaction of the fluid accelerated by the motion of a solid body, or "apparent mass" forces; and

(*c*) Forces that depend on the vorticity distribution in the wake.

The forces in group (*a*) are readily obtained by wind-tunnel tests of straight and curved models under steady flow conditions. The forces in group (*b*) are in phase with the inertia forces acting on the oscillating mass of the structure,

Note.—This paper by D. B. Steinman was published in October, 1949, *Proceedings*.

[25] Head, Dept. of Aeronautical Eng., Pennsylvania State College, State College, Pa.

[26] "Airfoil Theory for Non-Uniform Motion," by Theodor von Kármán and W. R. Sears, *Journal of the Aeronautical Sciences*, August, 1938, p. 379.

and are negligible in comparison for suspension bridges. The forces in group (*c*) may be evaluated theoretically for a thin airfoil, but a similar analysis for a suspension bridge cross section appears impossible.

The theoretical thin airfoil, shown in Fig. 24(*a*), has a lift force of $\pi\, \rho\, V^2\, b\, \alpha$ acting at the quarter chord point. Similarly, the curved thin airfoil in Fig. 24(*b*) has a lift force of $\pi\, \rho\, V^2\, b\, \beta/2$ acting at the midpoint. These theoretical forces differ slightly from experimental values because the airflow tends to separate from the leading edge of a thin plate in violation of the assumed condition of a rounded leading edge. For the conditions shown in Fig. 8(*b*) the theoretical pressures are negative at all points, and are symmetrically distributed as the ordinates of a semi-ellipse. The positive experimental values shown result from flow separation at the corner of the leading edge, and this condition also exists for practical bridge roadways.

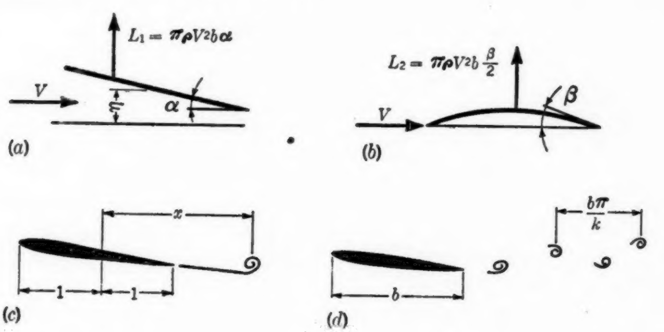

Fig. 24

For the oscillating thin airfoil, the effective angle of attack is decreased because of the vertical velocity $\dot{\eta}$. The vertical velocity varies across the width, and has a value of $\dot{\eta} - \frac{b\,\dot{\alpha}}{4}$ at the third quarter chord point, which determines the angle of attack of the airfoil. The effective angle of attack is therefore $\alpha - \frac{\dot{\eta}}{V} + \frac{b\,\dot{\alpha}}{4\,V}$. Substituting this value for the lift force in Fig. 24(*a*) the following value is obtained:

$$L_1 = \pi\, \rho\, V^2\, b \left(\alpha - \frac{\dot{\eta}}{V} + \frac{b\,\dot{\alpha}}{4\,V}\right) \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

The effective angular velocity at the midchord point is equal to $\dot{\alpha} - \frac{\ddot{\eta}}{V}$. From the author's discussion of curved models, this is equal to $\frac{2\,V\,\beta}{b}$, with β as shown in Fig. 24(*b*). The following lift force resulting from the effective angular velocity is obtained from Fig. 24(*b*), thus:

$$L_2 = \frac{\pi}{4}\, \rho\, V\, b^2 \left(\dot{\alpha} - \frac{\ddot{\eta}}{V}\right) \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

The last term of Eq. 42 represents the "apparent mass" effect, and is equal to the inertia force on a cylinder of air of diameter b. The other terms of L_1 and L_2 are quasi-steady forces, which the author obtains from wind-tunnel tests of static models similar to those shown in Figs. 24(a) and 24(b).

When an airfoil is suddenly moved so that the lift is changed, a vortex is shed from the trailing edge, and this vortex moves downstream with the fluid as shown in Fig. 24(c). The lift on the airfoil does not reach its final value until this vortex has moved an infinite distance downstream, and the lift is considerably smaller when the vortex is near the airfoil. The vortex trail behind an oscillating airfoil is shown in Fig. 24(d), and the reduction in lift, $-L_3$, resulting from this vortex trail, may be expressed in the following form:

$$-L_3 = (1 - C)\,L_1 \qquad (43)$$

The term C is evaluated by Theodore Theodorsen[2] by considering the vortex trail between the trailing edge ($x = 1$) and infinity; thus:

$$C = \frac{\displaystyle\int_1^\infty \frac{x\,e^{-ikx}\,dx}{\sqrt{x^2 - 1}}}{\displaystyle\int_1^\infty \frac{(x + 1)\,e^{-ikx}\,dx}{\sqrt{x^2 - 1}}} \qquad (44)$$

These integrals are evaluated as Bessel functions. Numerical values of $C = F + i\,G$ have been plotted for values of $k = \omega\,b/(2\,V)$ by Mr. Theodorsen, and have been reproduced by Friedrich Bleich,[27] M. ASCE.

The total lift force is obtained as the sum of the components, L_1, L_2, and L_3, as follows:

$$L = \pi\,\rho\,V^2\,b\,C\left(\alpha - \frac{\dot{\eta}}{V} + \frac{b\,\dot{\alpha}}{4\,V}\right) + \frac{\pi}{4}\,\rho\,V\,b^2\left(\dot{\alpha} - \frac{\ddot{\eta}}{V}\right) \qquad (45)$$

Eq. 45, obtained by combining the steady flow conditions of Figs. 24(a) and 24(b) with the wake effects of Fig. 24(d), corresponds with the values obtained by Mr. Theodorsen and other investigators from considerations of the velocity potential for unsteady flow conditions. This verifies the author's procedure of using static models to obtain the quasi-steady forces.

The forces resulting from the wake vorticity will be considerably different for a bridge cross section than for an airfoil. Eqs. 43 and 44 are derived for an airfoil by assuming that the Kutta condition, in which the flow leaves the sharp trailing edge, is satisfied at any instant. For an oscillating bridge cross section with a blunt trailing edge, the trailing vortices may be shed at various points. The author has evaluated the wake effects from the pressure distribution on a static model, considering a phase lag. This procedure involves more approximations than were used in obtaining the quasi-steady forces, but appears to be the best method available for evaluating these effects.

At present, two methods are available for calculating aerodynamic forces on oscillating bridge sections. Mr. Bleich[27] uses the basic forces for a thin airfoil

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[2] "General Theory of Aerodynamic Instability and the Mechanism of Flutter," by Theodore Theodorsen, *Technical Report No. 496*, National Advisory Committee for Aeronautics, Washington, D. C., 1935.

[27] "Dynamic Instability of Truss-Stiffened Suspension Bridges Under Wind Action," by Friedrich Bleich, *Transactions*, ASCE, Vol. 114, 1949, p. 1177.

and superimposes an alternating force that must be determined empirically from dynamic model tests. This method applies only to open truss structures which approximate a flat plate. The author determines all forces from static model tests, and thus obtains a solution which is applicable to all cross sections, including girder-stiffened roadways.

In reviewing the present state of knowledge of suspension bridge vibration analysis, it appears that a designer who makes use of the available theory may be confident of the safety of his structure. Static models should be tested carefully. The models should span between walls of the tunnel to insure two-dimensional flow. Since flow separation effects are important, the Reynolds number for the model flow should correspond with that for the prototype. Tests on a dynamic model of the final structure are probably advisable in order to verify the calculations of aerodynamic forces.

Since aerodynamic forces cannot be obtained exactly, some conservatism in the design for wind forces is desirable. However, it seems overconservative to revert to the heavy types of stiffening trusses which were used in the early 1900's. If such conservatism is substituted for scientific analysis, forty years of progress in suspension bridge design will have been lost.

ALEXANDER KLEMIN.[28]—By sheer virtue of necessity, aeronautics leads the field in its knowledge of aerodynamics and in its powerful methods of attacking problems of oscillation. For example, in the study of wing flutter, the aeronautical engineer considers the following factors: Mass, moment of inertia, elastic resistance to distortion, gravitational resistance to displacement, internal friction of the material, aerodynamic forces, and damping moments and their changes with vertical and angular velocities. He must make full use of the wind tunnel, in force and oscillation tests, using specially constructed models which follow the laws of dynamic similarity. When he has written down the long, complicated equations, he finds that they cannot be solved because aerodynamic forces vary as the square of the velocity, and the differential equations are no longer (alas) linear differential equations with constant coefficients. He then introduces the powerful idea of "resistance derivatives," which render the equations tractable. When attempts are made to deal with aeronautical oscillation problems by "short-cut methods," these methods fail. It is apparently essential to make a complete, recondite analysis, to take all factors into account, and to study "coupling," which is the bugbear of electrical as well as of aeronautical engineering.

From the writer's reading of the subject of bridge oscillations, he has the impression that civil engineers have tried to solve such problems by simplified methods, without sufficient aerodynamic data, and have failed.

The great merit of this paper lies in the fact that the author has boldly, and with originality, adapted aeronautical methods, both experimental and analytical, to the problem of bridge oscillations. The writer is satisfied that the methods which proved sound in aeronautics have been correctly and carefully adapted in this paper. Thus, perhaps for the first time in engineering literature, a paper has provided a sound basis for the study of bridge oscillations.

[28] Helicopter Editor, *Aero Digest*, Editor, International Handbook of Aeronautical Eng., Greenwic Conn.

ABRAHAM SLAVIN,[29] M. ASCE.—The subject matter in this paper is essentially related to aerodynamic stability of bridges, although it is completely generalized for applicability to other types of section. The author had previously developed simple criteria and formulas for determining the aerodynamic stability of suspension bridges.[30] These simple criteria are of particular significance and considerable practical value.[31,32,33] The paper under discussion is a continuance of this pioneering work and a notable contribution to engineering science. It represents creative research work in solving a new and critical problem with which the profession has been dramatically confronted. For full appreciation of the analytical and practical significance of this important contribution, it should be recognized that it is the first complete theory of aerodynamic oscillations which has the following pertinent features:

(*a*) It is not limited to streamline airfoil sections and idealized thin plates but is applicable to all bridge sections of whatever type or form, including flat plates, girder-stiffened sections, truss-stiffened sections, and all modified or combination sections;

(*b*) It is not limited to coupled oscillations (as in airplane flutter) but also covers vertical oscillations and torsional oscillations (which are the known forms of bridge instability);

(*c*) It is not limited to determining critical flutter velocity but also determines and predicts critical velocities and, in addition, negative damping, rate of amplification, limiting amplitudes, and amplitude response at all wind velocities;

(*d*) It does not require oscillating model tests, which are costly and time consuming, but instead determines all necessary constants of any section by simple static tests on small-scale section models; and

(*e*) It is independent of the known defects and inconsistencies of conventional aerodynamic theory.

Those who have sought to apply conventional airfoil theory to bridge sections have overlooked the fact that the conventional theory assumes rounded leading edges and sharp, tapering trailing edges, and any attempt to apply the theory to bridge sections or even to thin flat plates introduces material errors in the critical region in the vicinity of the leading edge. It should be evident that such reasoning, although perhaps excellent for mathematical exposition, does not apply to the structural sections in bridges. The nonstreamline sections in bridges do not fit the classical derivation, in aerodynamic theory, of the behavior of the streamline sections.

In this paper, the author has combined aerodynamic science, vibration theory, and bridge dynamics, to create a new scientific approach to the problem of bridge aerodynamics. The resulting contribution is a technical achievement that is of the highest practical usefulness.

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[29] Cons. Civ. Engr. and Architect, New York, N. Y., and with the Dept. of Civ. Eng., Polytechnic Inst. of Brooklyn, Brooklyn, N. Y.

[30] "Rigidity and Aerodynamic Stability of Suspension Bridges," by D. B. Steinman, *Transactions*, ASCE, Vol. 110, 1945, p. 439.

[31] *Ibid.*, pp. 572–574.

[32] *Ibid.*, p. 575, Eq. 177.

[33] *Ibid.*, p. 576, Eq. 178.